AD-A101 711 HOWARD NEEDLES TAMMEN AND BERGENDOFF NEW YORK F/G 13/13
ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR, COY GLEN AND CA--ETC(U)
AUG 75 DACW49-75-C-0052

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1 OF 3
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Lehigh Valley Railroad
Coy Glen
Riprap Repair

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The design analysis for this project provides detailed designs in four sections for the following items: 1, Two hydraulic drop structures and attached wingwall on Coy Glen, 2. Soils and foundation analysis for the above structures and cantiliver sheet pile wingwall alternates for the two drop structures, 3) Riprap repair for the section in Cayuga Channel between the Lehigh Valley Railroad bridge and the drop structure at Station 160+00, 4, Dynamic water loads on the drop stucture and hydraulic design for Coy Glen by the Buffalo District. The design considered two types of wingwalls. The factor of

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safety in bearing for the concrete wingwalls is not considered adequate and the more conservative steel sheet pile wingwalls are recommended.

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DESIGN ANALYSIS

ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR

COY GLEN AND CAYUGA INLET

ITHACA, NEW YORK

CONTRACT NO. DACW49-75-C-0052

WORK ORDER NO. 1

DEPARTMENT OF THE ARMY

BUFFALO DISTRICT, CORPS OF ENGINEERS

AUGUST 1975

PREPARED BY

HNTB

HOWARD NEEDLES TAMMEN & BERGENDOFF

DESIGN ANALYSIS

ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR

COY GLEN AND CAYUGA INLET

ITHACA, NEW YORK

INDEX

DESIGN ANALYSIS

ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR

COY GLEN AND CAYUGA INLET

ITHACA, NEW YORK

CONTRACT NO. DACW49-75-C-0052

WORK ORDER NO. 1

SCOPE & GENERAL RECOMMENDATIONS

The design analysis for this project provides detailed designs in four sections for the following items.

1. Two hydraulic drop structures and attached wingwalls on Coy Glen.

2. Soils and foundation analysis for the above structures and cantiliver sheet pile wingwall alternates for the two drop structures.

3. Riprap repair for the section in Cayuga Channel between the Lehigh Valley Railroad bridge and the drop structure at Station 160+00.

4. Dynamic water loads on the drop structure and hydraulic design for Coy Glen by the Buffalo District.

The design considered two types of wingwalls. The factor of safety in bearing for the concrete wingwalls is not considered adequate and the more conservative steel sheet pile wingwalls are recommended.

1. DROP STRUCTURE DESIGN

1.1 This design is for the two drop structures and concrete cantilever wingwalls of Coy Glen.

1.2 The design of the drop structures was for two limiting loading conditions: (a) no flow (empty) with saturated soil; and (b) design flow with dynamic hydraulic impact. The hydraulic loads are based on a 50-year design flow.

1.3 The walls for the drop structure are designed for an at-rest earth pressure plus water pressure. Calculations for the wall design are on Sheets S-4 to 20.

1.4 At-rest lateral earth pressure plus water pressure was used for the design of the end sills. The calculations are on Sheets S-21 to 28.

1.5 The baffle flocks in the bottom of the drop structures have been designed for a horizontal hydraulic dynamic force of 3,000 pounds each. The calculations are on Sheet S-29.

1.6 The bottom slab of the drop structures has been designed for normal dead load plus a vertical hydraulic dynamic load of 1,630 p.s.f. over a five-foot by 15-foot area. It was also checked against uplift from ground water pressure. The calculations are on Sheets S-30 to 42.

1.7 The wingwalls for the drop structure were designed for an active earth pressure plus water pressure. Three wall heights were designed, one for the downstream end of the structures and two for the upstream end of the structures. For the latter two wall designs one is founded at the same level as the drop structure and the other is founded five feet above this level. Calculations are on Sheets S-43 to 58.

1.8 Detailed plans, elevations, sections and construction procedures have been developed for the drop structure and the concrete cantilever wingwall alternates. These data are given on Sheets S-59 to 66.

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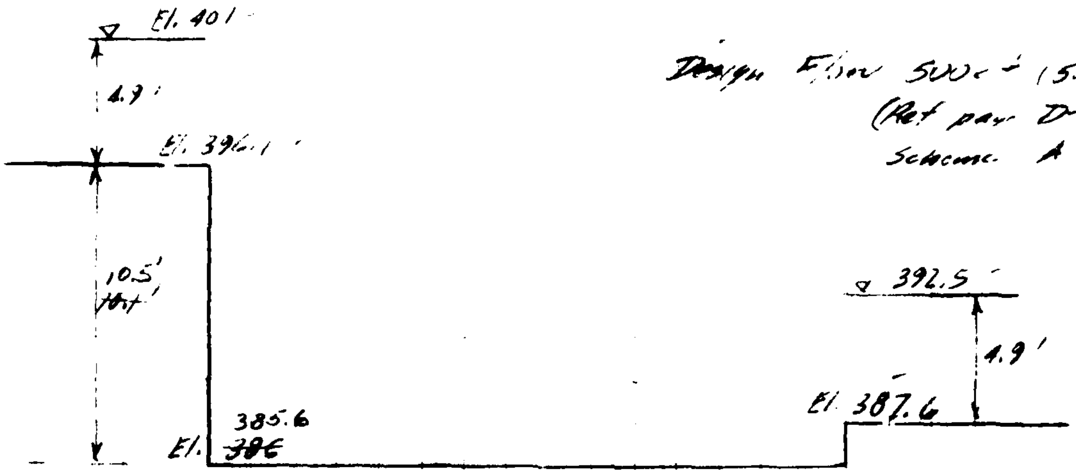
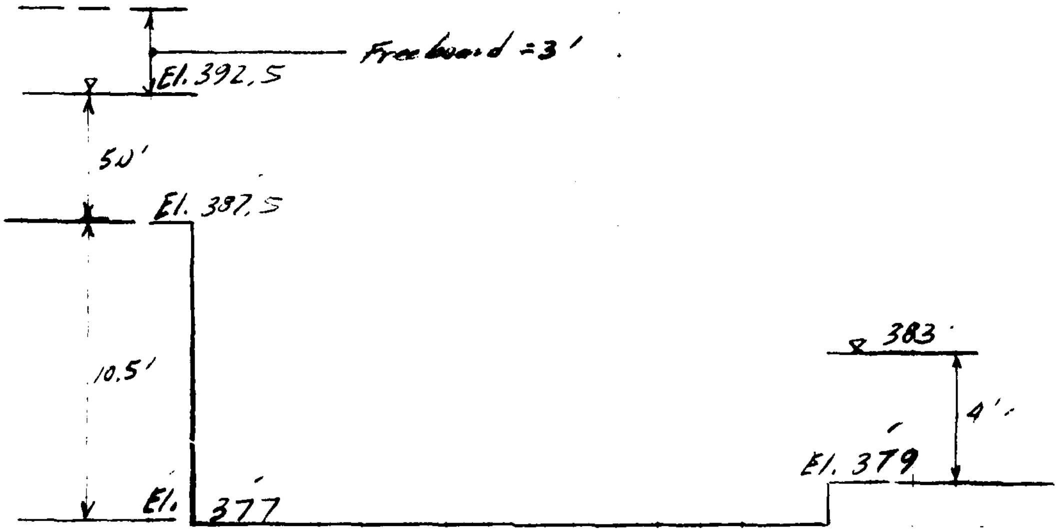
SHEET NO. S-1

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

Design Flow 500c.f.s. (50 yr)
(Ref par D-1)
Scheme A



Str. No. 1 (Incl. D-2)



Str. No. 2 (Incl. D-2)

For Structure Geometry use Scheme 1A from Enclosure D-1 (Ref. 1.1, sh. D-1)
Allow 3' Freeboard (Encl. D-2)

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CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

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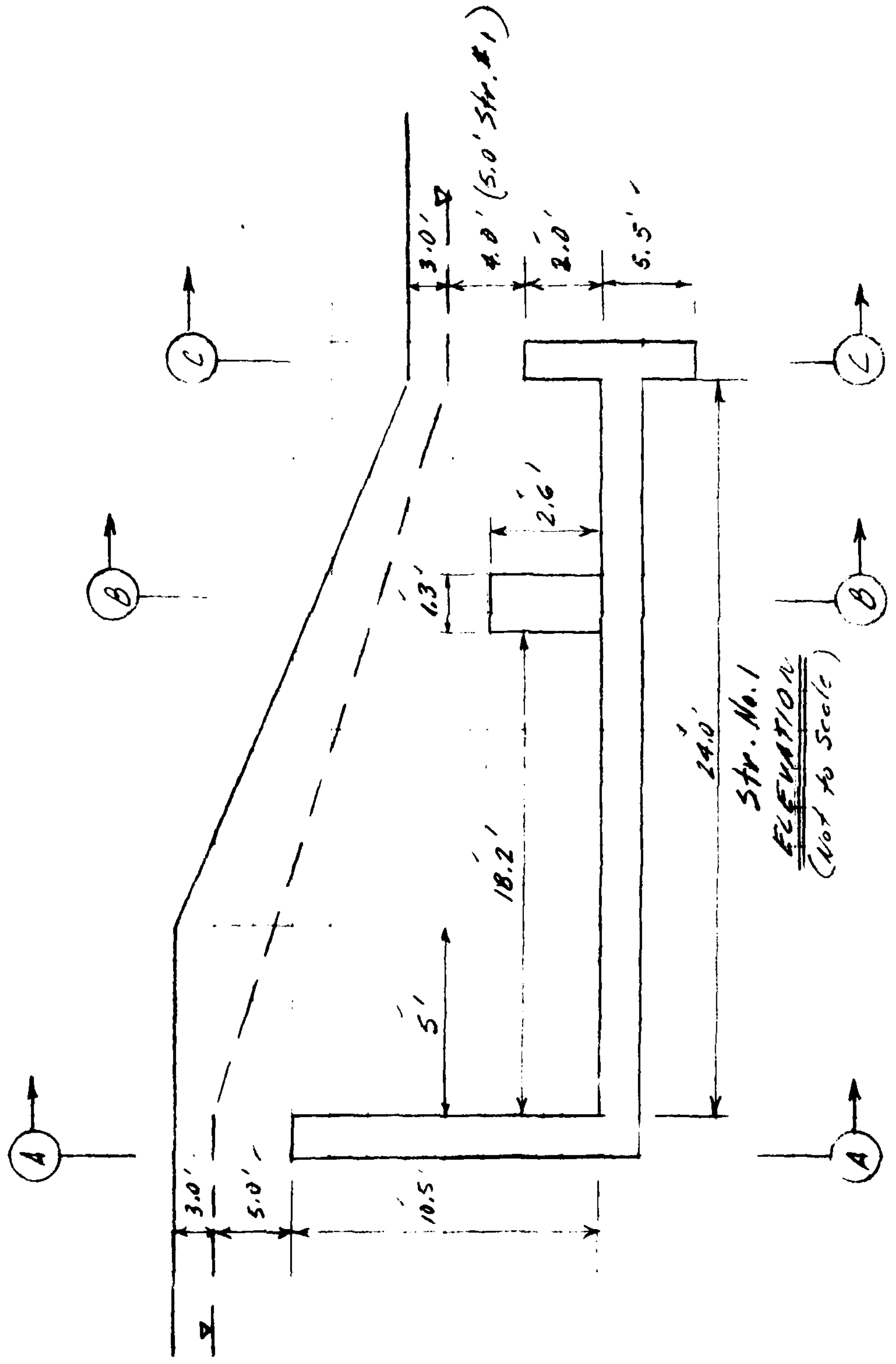

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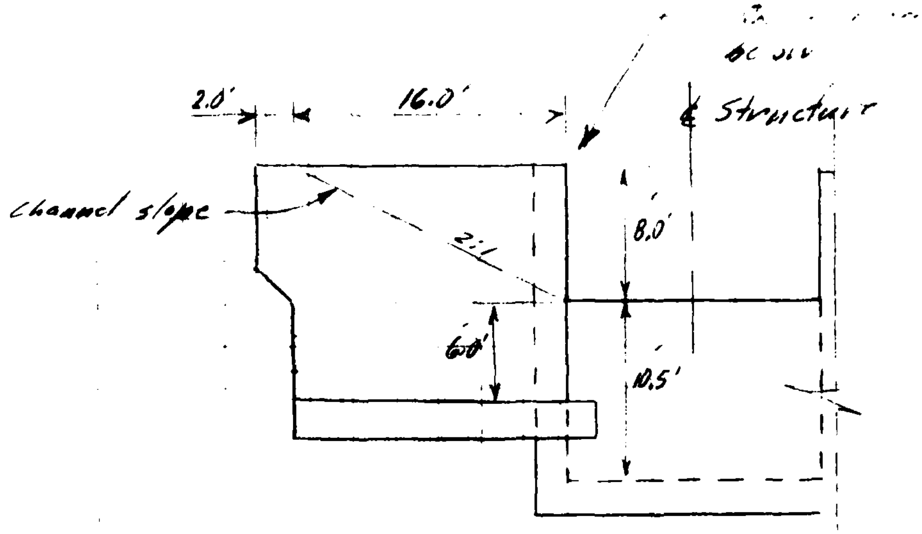
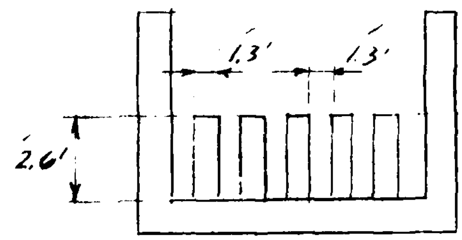
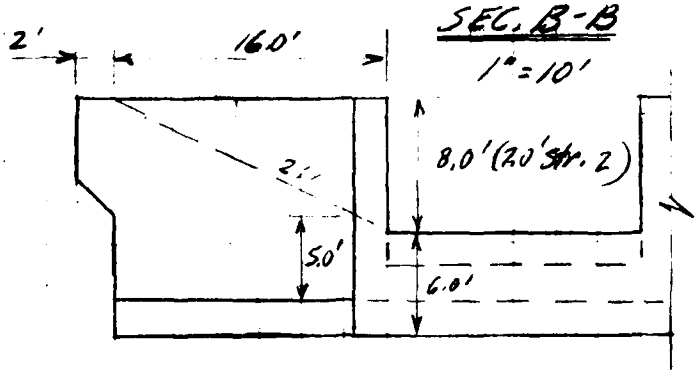
CALCULATIONS FOR Coy Glenn, Ithaca, N.Y.



SEC. A-A
1" = 10'



SEC. B-B
1" = 10'



SEC. C-C
1" = 10'

Rounding at Inlet Wall

$r = 0.6\, d_c$ (per R. Gorecki - Buffalo Dist, 4-2-75)

$d_c = \left(\frac{q^2}{g}\right)^{1/3}$ per "Design of Small Dams" USBR, 2nd Ed, 1973 p 410

$d_c = \left[\frac{(500/15)^2}{32.2}\right]^{1/3} = 3.26'$; $r = 0.6 \times 3.26 = 1.96'$ use $2.0'$ .6

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CALCULATIONS FOR ... Glen ... N.Y.

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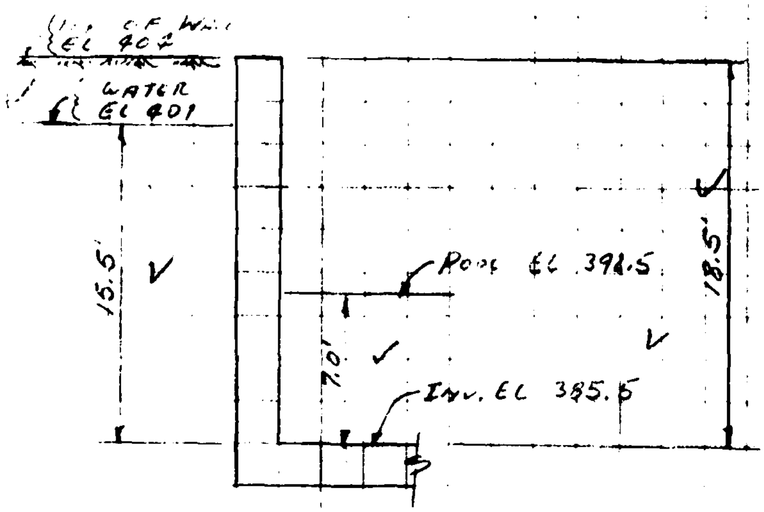
STR No. 1 Box



FOR EL. SEE H,N,T&B. SH No. 1

HORIZ ... PRES ... = H.G.F.

SAT. UNIT WT. ~ p_s = 68.5 #/ft ... H,N,T&B. ...
BUOYANT ... PR ~ p_b = 35.3 #/ft ... No. A1
UNIT WT OF WATER ~ p_w = 62.4 #/ft

CASE a) ~ NO FLOW (EMPTY) WITH SAT. SOIL

CASE b) ~ DESIGN FLOW WITH DYNAMIC HYD ...

CASE a + b FROM RESUME OF ...

CASE a + b ARE GROUP I LOADING @ 100%
REF. EM 1110-1-... PARA. 4

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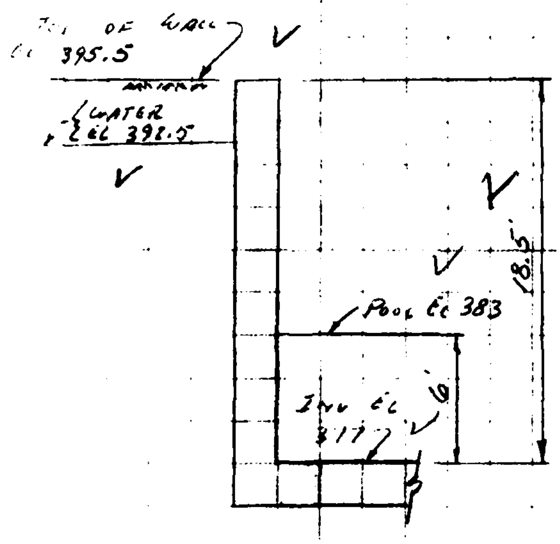
CALCULATIONS FOR Cox Glen, [illegible], N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

STR No 2 Box



For El see H.N.T.B. Sht No 1

CRITICAL DESIGN COND & SECTIONS

CASE a) Both Str No 1 & 2 are similar since the heights [illegible] equal, [illegible] M.C.P. are equal

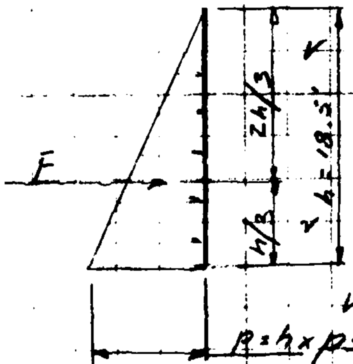
CASE b) Since the height [illegible] 18.5' [illegible] in all structures and pool depth is changed in structure No 2, it is obvious that structure No 2 [illegible] chosen for design. (6' Pool Depth)

CASE b') Also investigate Case b with 2' Pool Depth. This condition [illegible] [illegible] wall design.

CALCULATIONS FOR Coy Glen, Ithaca, NY

MAX. WALL MOM.

Case a) Str. No 1 & 2



p = h x ps = 18.5 x 66.5 = 1230 #/'

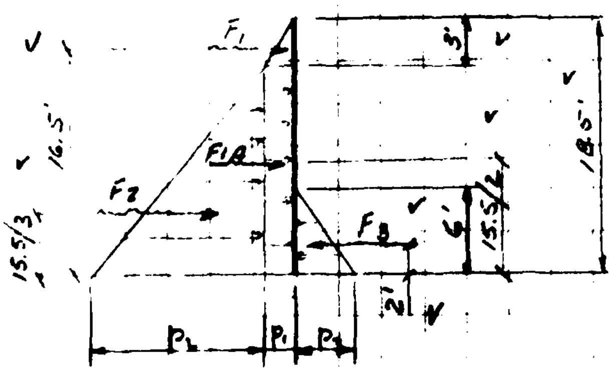
Result. Force = F acting h/3 above base

F = p x h ÷ 2 = 1230 x 18.5 ÷ 2 = 11380 #

Mom = F x h ÷ 3 = 11380 x 18.5 ÷ 3 = 70,180 '#

Not Gov. Mom.

Case b) Str. No 2



p1 = Press. Due to Sat. Earth
p2 = " " " Water + Buoyant Earth
p3 = Press. Due to Flowing Water in Box

$p_1 = h \times p_s = 3 \times 66.5 = 200 \#/'$

$p_2 = h \times (p_b + p_w) = 15.5 (35.3 + 62.4) = 1514 \#/'$

$p_3 = h \times p_w = 6 \times 62.4 = 375 \#/'$

$F_1 = 200 \times 3 \div 2 = 300$ ✓

$F_{1A} = 200 \times 15.5 = 3,100$ ✓

$F_2 = 1514 \times 15.5 \div 2 = 11,735$ ✓

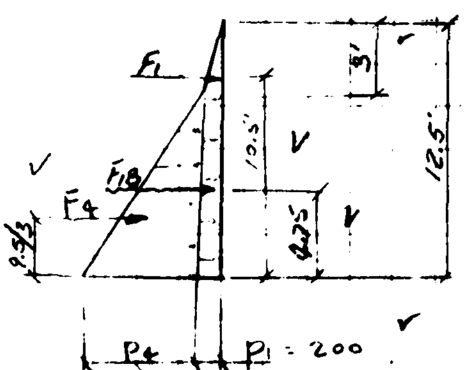
$F_3 = 375 \times 6 \div 2 = -1,125$ ✓

$\Sigma F = 14,010 \#$ ✓

$Mom = 300 \times 16.5 + 3100 \times \frac{15.5}{2} + 11,735 \times \frac{15.5}{3} - 1125 \times 2 =$

$= 87,355 '\#$ @ Top of Ftg

Find Max Mom 6' above Inv



$P_4 = 9.5(35.3 + 62.4) = 928$

$F_1 = 200 \times 3 \div 2 = 300$ ✓

$F_{1B} = 200 \times 9.5 = 1900$ ✓

$F_4 = 928 \times 9.5 \div 2 = 4410$ ✓

$\Sigma F = 6,610 \#$ ✓

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SHEET NO. 5-8

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

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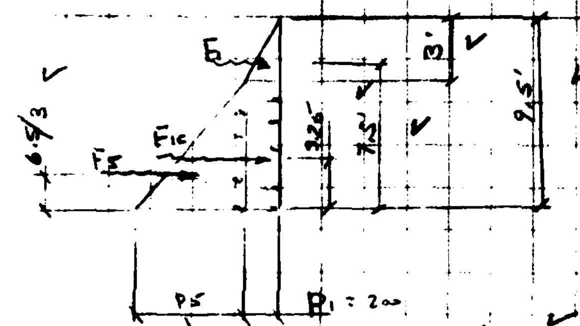
Mom = 300 × 10.5 + 1900 × 4.75 + 4410 × 9.5/3 =

= 26,140'# 6' below top of fig

Find Max Mom 9' above Inv.



P5 = 6.5(35.3 + 62.4) = 635#/

$F_1 = 200 \times 3 \div 2 = 300$

$F_{1c} = 200 \times 6.5 = 1300$

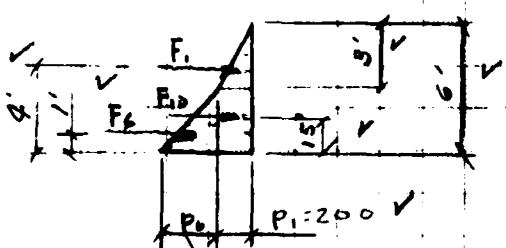
$F_5 = 635 \times 6.5 \div 2 = 2065$

$\Sigma F = 3665\#$

Mom = 300 × 7.5 + 1300 × 3.25 + 2065 × 6.5/3 =

= 10,950'# 9' above top of ftg

Find Max Mom 12.5' above Inv.



P6 = 3(35.3 + 62.4) = 293#/

$F_1 = 200 \times 3 \div 2 = 300$

$F_{1D} = 200 \times 3 = 600$

$F_6 = 293 \times 3 \div 2 = 440$

$\Sigma F = 1340\#$

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CALCULATIONS FOR

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SHEET NO. 5-9

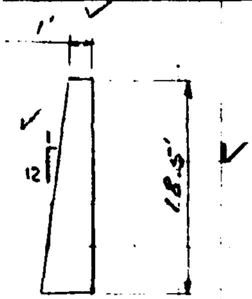
Mom = 300×4 + 600×1.5 + 440×1 =

= 2,540'# @ 3' Above Top of Ftg.

Find Max Mom 15.5' Above Ftg.

Mom = F_1 × 3 ÷ 3 = 300 × 3 ÷ 3 = 300'#

Find Wall Thickness @ Varied Heights



@ Top of Ftg t = 12" + 18.5×12÷12 = 30.5"

6' Above Top of Ftg t = 30.5 − 6×12÷12 = 24.5"

9' do. t = 30.5 − 9×12÷12 = 21.5"

12.5' do. t = 30.5 − 12.5×12÷12 = 18.0"

15.5' do t = 30.5 − 15.5×12÷12 = 15.0"

2.0 do t = 30.5 − 2×12÷12 = 28.5"

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MADE BY C.D. DATE 4/26/75 JOB NO. 4C Y
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SHEET NO. 5-7H

CALCULATIONS FOR Coy Glen Ithaca, N.Y.

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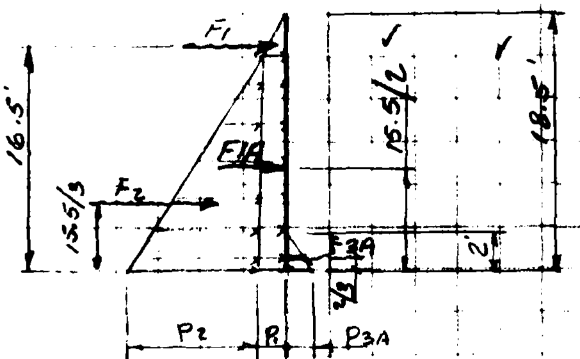
CASE 5') STR NO 2



P1 = PRESS. DUE TO SAT EARTH
P2 = " " " WATER & BUOYANT EARTH
P3A = PRESS. DUE TO 2' POOL OF WATER IN BOX.

P1 = 200 #/1 }
P2 = 1514 #/1 } See Sh. No 7

P3A: 2 x 62.4 = 125 #/1

F1 = 300 # }
F1A = 3,100 # } See Sh No 7
F2 = 11,725 # }

F3A = -125 = (125 x 2 ÷ 2)

ΣF = 15,100 #

Mom = 300 x 16.5 + 3100 x 15.5/2 + 11,725 x 15.5/3 -

-125 x 2/3 = 89,520 '# @ Top of FTG

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SHEET NO. 5-7B

CALCULATIONS FOR Bx Green, Ithaca, N.Y.

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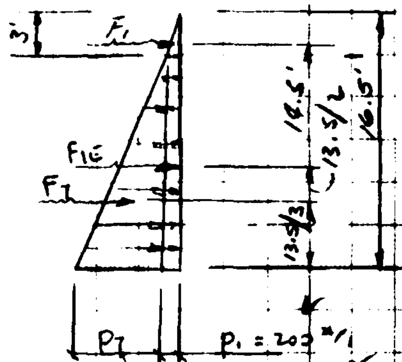
Find Mom 2' Above Inv.



$P_7 = 13.5(35.3 + 62.4) = 1319$ #/'

$F_1 = 300$#

$F_{1E} = 2700$# $= 13.5 \times 200$

$F_7 = 8905$# $= 1319 \times 13.5 \div 2$

$\Sigma F = 11905$#

$Mom = 300 \times 14.5 + 2700 \times \frac{13.5}{2} + 8905 \times \frac{13.5}{3} =$

$= 62,650$ '# 2' above top of inv.

Mom 5', 12.5' & 15.5' above invert are the same as previously [illegible] on sh 8 & 9.

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SHEET NO. 5-10

CALCULATIONS FOR Coy Geor. I WARN

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

f'c = 3000 psi (Pg 3 # 6 EM 1110-1-2101)

fc = .35 x f'c = 1050 psi (Pg 4 # 7a dp)

GRADE 40 STEEL fy = 40,000 psi (Pg # 4 EM 1110-2-2103)

fs = 20,000 psi (Pg 4 # 7a EM 1110-1-2101)

Min. Cl. Cov = 4" (Pg 2 # 3 EM 1110-2-2103)

j = .891 (Reinf. Conc. Design Handbook) ACI SP-3 Table 1

Assume Cl. for #10 Bars

$4'' + 1.27/2 = 4.64''$

$\therefore d = t - 4.64''$

$d_{BASE} = 30.5 - 4.64 = 25.86''$
$d_{2'} = 28.5 - 4.64 = 23.86$
$d_{6'} = 24.5 - 4.64 = 19.86''$

$d_{9'} = 21.5 - 4.64 = 16.86''$

$d_{11.5} = 18.0 - 4.64 = 13.36''$

$d_{14.5} = 15.0 - 4.64 = 10.36''$

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SHEET NO. 5-11

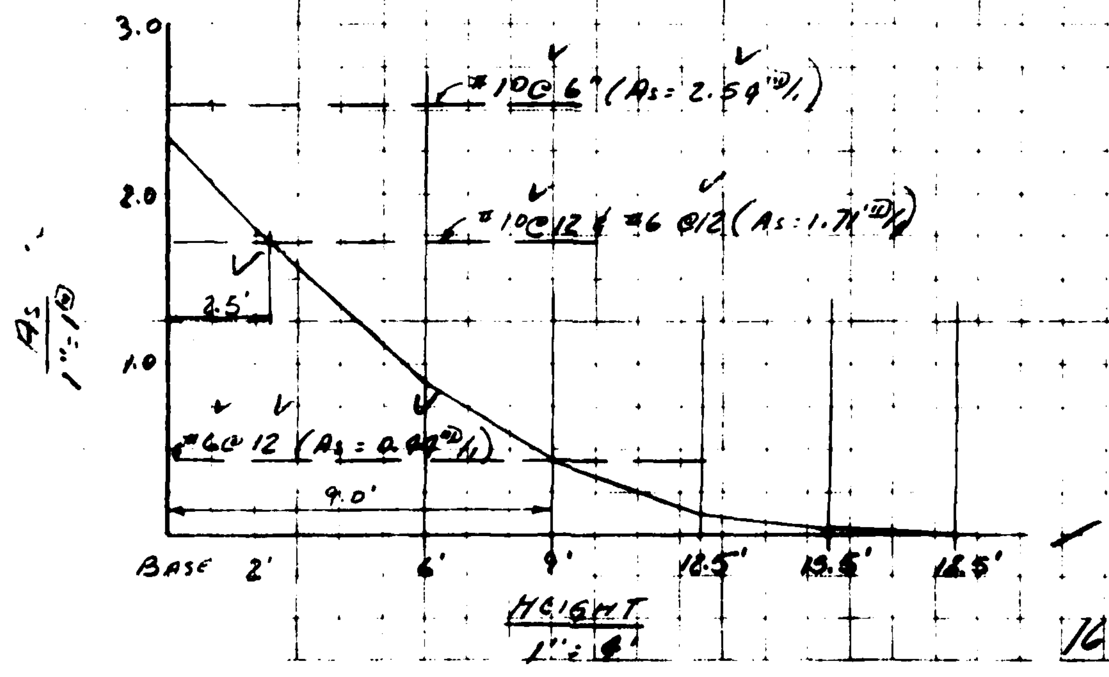
CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

$As = M \div fs \times j \times d$

As = Area of Reinf. Steel Req'd (in²/ft)
M = Design Mom. (in k/ft)
fs = Allow. Stress in Reinf. Steel (k/in²)
j = Ratio of Dist. (jd) between Resultants of Compressive & Tensile Stresses to Effective Depth.
d = Effective Depth (in)

As base	89.52 × 12 ÷ 20 × .891 × 25.86	= 2.33 in²/1
As 2'	= 62.05 × 12 ÷ × 23.86	= 1.77 in²/1
As 6'	= 26.10 × 12 ÷ × 19.86	= 0.83 in²/1
As 9'	= 10.95 × 12 ÷ × 16.36	= 0.44 in²/1
As 12.5'	= 2.54 × 12 ÷ × 13.36	= 0.13 in²/1
As 15.5'	= 0.30 × 12 ÷ × 10.36	= 0.02 in²/1



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MADE BY ___ DATE 4/7/75 JOB NO. 4204
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SHEET NO. S-12

CALCULATIONS FOR Coy Glen Ithaca, N.Y.

Dev. Length of Deformed Bars (ACI Std 318-71) (# 12.15 Pg 43)

$0.04 \times A_b f_y / (f'_c)^{1/2}$

#5	~ 0.04 × .31 × 40,000 ÷ (3000)^½	= 9.1"	
#6	.44	= 12.9"	← Gov
#7	.60	= 17.5"	← Gov
#8	.79	= 23.1"	← Gov
#9	1.0	= 29.2"	← Gov
#10	1.27	= 37.1"	← Gov
#11	1.56	= 45.6"	← Gov

$0.0004 \times d_b \times f_y$

#5	~ 0.0004 × .625 × 40,000	= 10.0"	← Gov
#6	.750	= 12.0"	
#7	.875	= 14.0"	
#8	1.000	= 16.0"	
#9	1.128	18.0"	
#10	1.270	20.3"	
#11	1.410	22.6"	

* ACI STD 318-71 Pg 44 # 12.5(a)
** " " " Pg 18 # 7.6

Splice Length of #10 = .8* × 1.3** × 37.1 = 38.6" (use 39")

" " " #6 = 1.3** × 12*** = 15.6" (use 16")

*** Since 8 < 12.5 < 12" use 12" (ACI Pg 44 #12.5)

Length of #10 Dowel along Ftg = (2.5' = 30") +
+ 39" = 69" (use 6'-0")

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HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

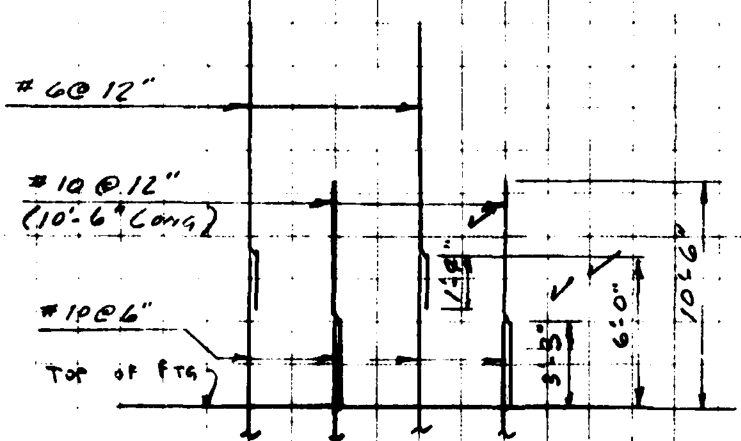
Determine where only #6 are Reqd.

ACI SP 319-71 Pg 42 # 12.1.4

d @ 9' = 16.86" (see Sh 11) ← Gov.

$12\phi = 12 \times 1.27 = 15.24$

End #10 = 9.0 (Sh 11) + 16.86/12 = 10.41' Above Inv.



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MADE BY C.D. DATE 4/7/75 JOB NO. 4234

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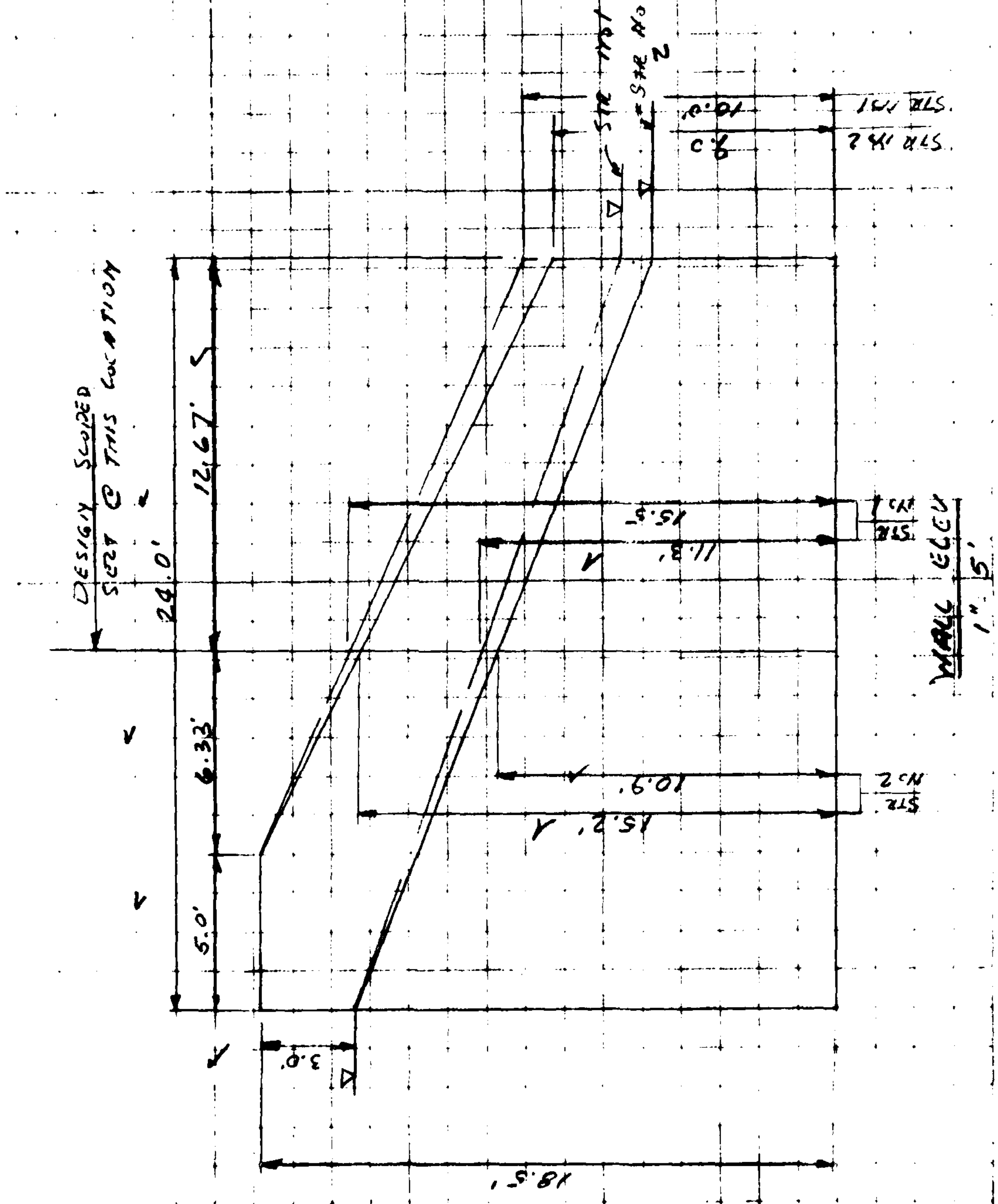
SHEET NO. 5-14

CALCULATIONS FOR Cay Glen, Ithaca, N.Y.

Design sloped section of B & wall @ 2/3 of its max height. From elev. on this sheet it can be seen that Str No 2 governs design, since water height in pool is 6' deep.



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CALCULATIONS FOR Coy Glen Ithaca, N.Y.

MADE BY C.D. DATE 4/11/75 JOB NO. 7604
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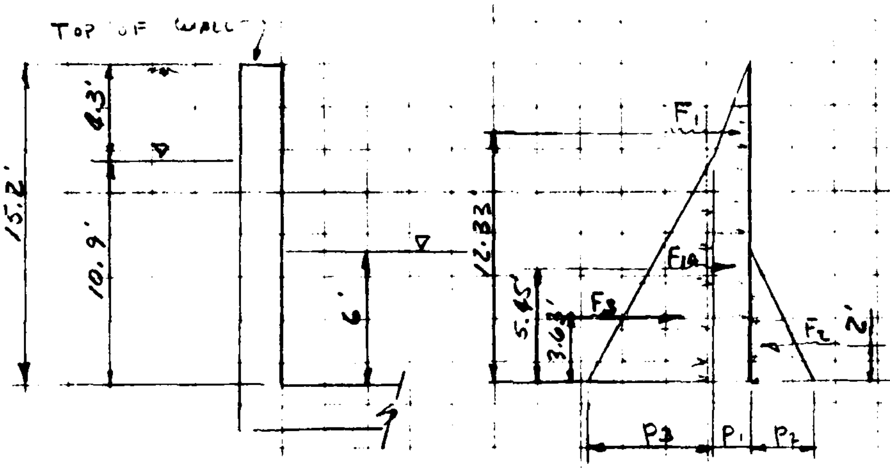
SHEET NO. 5-15

Case b) STR. No 2



$p_1 = h \times p_s = 4.3 \times 66.5 = 285$ #/'

$p_2 = h \times p_w = 6 \times 62.4 = 375$ #/'

$p_3 = h \times (p_b + p_w) = 10.9 \times (35.3 + 62.4) = 1065$ #/'

$F_1 = 285 \times 4.3 \div 2 = 615$

$F_{1A} = 285 \times 10.9 = 3,110$

$F_2 = 375 \times 6 \div 2 = -1,125$

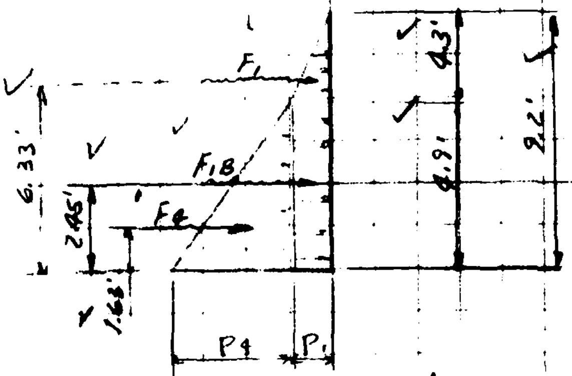
$F_3 = 1065 \times 10.9 \div 2 = 5805$

$\Sigma F = 8,405$ #

M at BASE $= 615 \times 12.33 + 3110 \times 5.45 + 5805 \times 3.63 - 1125 \times 2 = 43,355$ '#

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

FIND M [illegible] 6' ABOVE [illegible]



$p_1 = h \times p_s = 4.3 \times 66.5 = 285$ #/

$p_4 = h \times (p_s + p_w) = 4.9 (35.3 + 62.1) = 480$ #/

$F_1 = 285 \times 4.3 \div 2 = 615$

$F_{1B} = 285 \times 4.9 = 1395$

$F_4 = 480 \times 4.9 \div 2 = 1175$

$\Sigma F = 3185$ #

M [illegible] $= 615 \times 6.33 + 1395 \times 2.45 + 1175 \times 1.63$

$= 9225$ '#

SINCE As REQD [illegible] [illegible] [illegible] [illegible] ONLY TWO POINTS [illegible] [illegible] As / HEIGHT DIAGRAM. ([illegible] [illegible] [illegible] CONSERVATIVE)

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

CASE 6') STR. No. 2

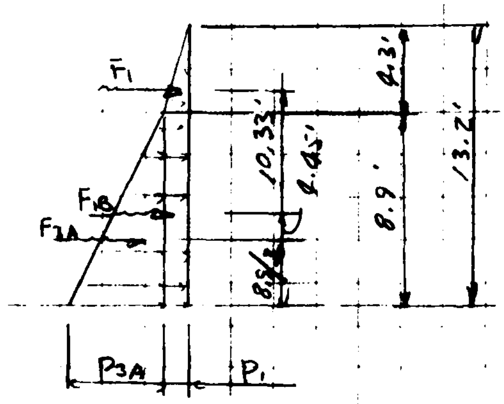
Adjust Mom with 6' Pool Depth to 2' Pool Depth

SH NO 15 — 6' Pool — 2' Pool

Mom @ Base = 43,355'# + 1125# × 2' − 2² × 62.4 × 1/3 =

= 45,520'#

Find Mom 2' Above Inv



p_1 = 285#/' See SH 15

p_{3A} = 8.9(35.3 + 62.4) = 870#/'

F_1 = 615 See SH 15

F_{1B} = 285 × 8.9 = 2,535#

F_{3A} = 870 × 8.9 ÷ 2 = 3870#

Mom 2' up = 615 × 10.33 + 2535 × 4.45 +

+ 3870 × $\frac{8.9}{3}$ = 29,115'#

HNTB MADE BY 1.W DATE 7/7/2? JOB NO. 4104
CHECKED BY J K J DATE 4/1/75 SEC. NO.
SHEET NO. 5-17

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

Assume Cover for #8 Bar

$4" + 1.0/2 = 4.5"$

$d = t - 4.5"$

$d_{BASE} = 12 A(15.2 \times 12 \div 12) - 4.5 = 22.7"$

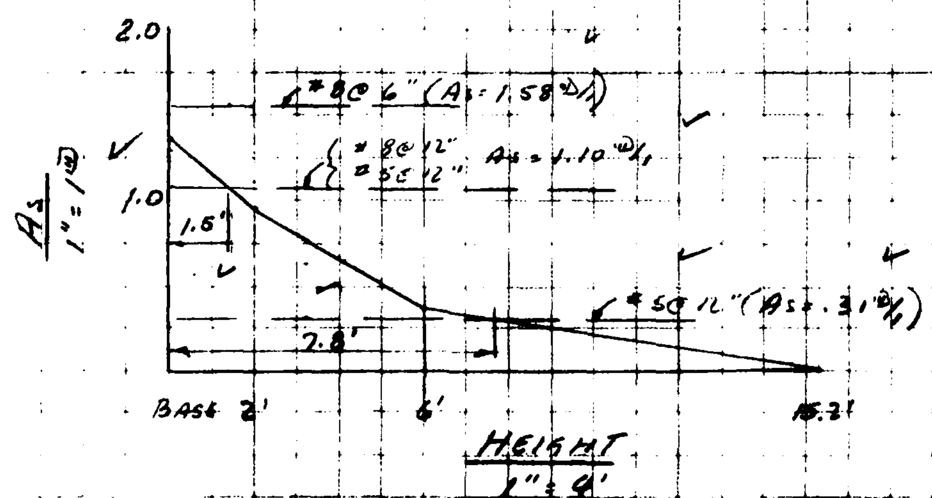
$d_{2' up} = 22.7 - (2 \times 12 \div 12) = 20.7"$

$d_{6' up} = 22.7 - (6 \times 12 \div 12) = 16.7"$

A_s BASE $= 45.56 \times 12 \div 20 \times .891 \times 22.7 = 1.35^{\square}"/'$

A_s 2' $= 29.12 \times 12 \div 20 \times .891 \times 20.7 = 0.95^{\square}"/'$

A_s 6' $= 9.23 \times 12 \div 20 \times .891 \times 16.7 = 0.37^{\square}"/'$



HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

HNTB

MADE BY C.L. DATE 1/1/75 JOB NO. 9204
CHECKED BY J.K.J DATE 4.17.75 SEC. NO.
SHEET NO. 5-18

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

H.N.T.&B. Sh No 12 and ACI 318-71 Sect 7-6 & 12-5

Splice Length of #8 = .8 x 23.1 x 1.3 = 24" (use 2'0")

∴ Make Short Dowel 2'-0" above Ftg > l_d = 23.1"

Splice Length of #5 = 1.3 x 12 = 15.6" say 1'4"

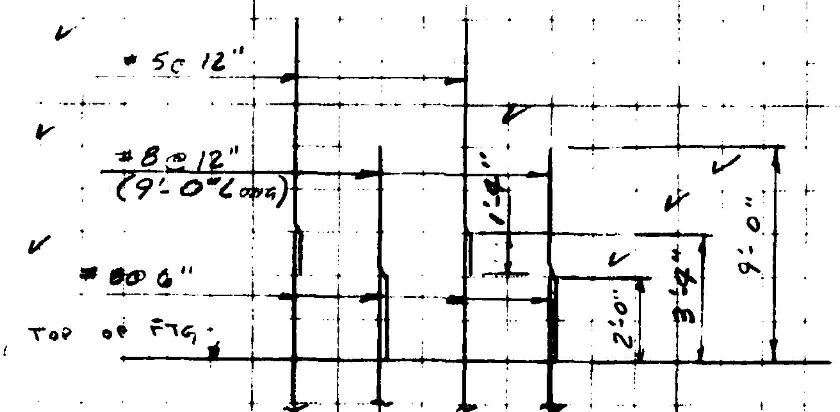
∴ Make Long Dowel 2'-0" + 1'-4" = 3'-4" above Ftg.

Location of ... #5 Reqd

ACI 318-71 { 12d = 12 x 1.0 = 12"

Sec 12.1.4 { $d_{7.8'}$: 22.7 - (7.8 x 12 ÷ 12) = 14.9" (use 1'3")

Height above Ftg = 7.8' + (1'-3") = 9.05' (use 9'-0")



HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Shear in Wall

Ref - ACI Building Code (318-71)
Pg 25 ¶ 8.10.3

Allow Shear @ 55% as given in sec

Pg 37 ¶ 11.4.1

$v_c = .55 \times 2 \times (f'_c)^{1/2} = .55 \times 2 \times (3000)^{1/2} = 60$ psi

To compute v_u (acting shear) in wall
see Pg. 41 ¶ 11.16.1

$v_u = V_u \div \phi h d$

ACI Pg 24 ¶ 8.10 $\phi = 1$

V_u = applied shear
$\phi = 1$
$d = .8\,l_w = .8 \times 12 = 9.6''$ (per ft)
h = thickness of section.

$\therefore v_u = V_u \div 1 \times h \times 9.6 = .104\,V_u \div h$

Sect @ top of ftg and previously given

For 18.5' Sect $v_u = .104 \times 14{,}010 \div 30.5 = 47.8 < 60$ ok
see sh 3 — see sh 9

see sh 15 — see sh 17

For Skewed Sect $v_u = .104 \times 9470 \div 27.2 = 36.2 < 60$ ok

HNTB CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

MADE BY L.D. DATE 4/9/75 JOB NO. 4004
CHECKED BY JKJ DATE 4.18.75 SEC. NO.
SHEET NO. S-20

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

TEMP + SHRINKAGE REINF IN BOX WALLS

EM 1110-2-2103 Pg 4, PARA 10

FOR HORIZ STEEL USE {10 b.(3) LOWER PORTION; 10 b.(1) UPPER PORTION}

RESTRAINED EDGE = 26' LONG

∴ 26 ÷ 4 = 6.5' OF HEIGHT

CROSS SECT. AREA @ HIGH END OF SLAB =

$(12 + 30.5) \times 18.5 \times \frac{12}{2} = 4718^{\square''}$

REQD AREA/FT = $\frac{4718}{2} \times \frac{.004}{18.5} = 0.51^{\square''}/'$

USE #7 @ 12" (A_s = .60 $^{\square''}/'$)

FOR UPPER PORTION A_s = .51/2 = .26 $^{\square''}/'$

USE #5 @ 12" (A_s = .31 $^{\square''}/'$)

FOR VERT STEEL USE 10 b.(1) (INS FACE)

USE AREA OF SECT. 2/3 DOWN FOR MIN HEIGHT

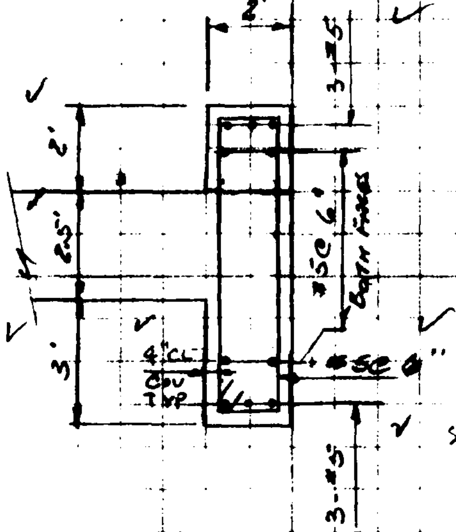
∴ THICKNESS = 12" + ($\frac{2}{3}$ × 18.5 × 12 ÷ 12) = 24.3"

STEEL AREA/FT = 12" × 24.3" × .002 ÷ 2 = .29 $^{\square''}/'$

SINCE THE WALL WILL BE SUBJECTED TO IMPACT AND TURBULENT FLOW USE #6 @ 12" WHICH IS MAX (A_s = .44 $^{\square''}/'$)

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

End Sill (Downstream)



Ref: EM 1110-2-2004

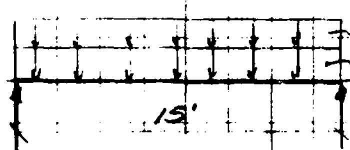
Assume no support from earth on downstream side

Say $d = 7'$ $\ell = 15'$

For Bot Reinf. Assume end sill is simply supported.

For Top Reinf. Assume end sill is fixed @ ends

Bot



Water = $2' \times 5' \times 62.4 = 624\#/'$

Beam = $2' \times 7.5' \times 150 = 2250\#/'$

$2,874\#/' = w$

$M = 2874 \times 15^2 \div 8 = 80,830'\# = 80.83'^k$

$V = 2,874 \times 15 \div 2 = 21,555\# = 21.6^k$

$A_s = 80.83 \div 20 \times .891 \times 7 = .65^{\square''}$ Use 3 #5 $A_s = .93^{\square''}$

Top: Since the design mom. will be smaller ($M = w\ell^2 \div 12$) use 3-#5 bars in top of end sill

HNTB

MADE BY C.T. DATE 7/9/7? JOB NO. 4204
CHECKED BY JAT DATE 4/18/75 SEC. NO.
SHEET NO. 5-22

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Check Shear

Ref ACI 318-71
* Pg 39 Para 11.9
** Pg 25 Para 8.10.3
*** Pg 37 Para 11.4.1

* $l_n \div d = 15 \div 7 = 2.14 < 5$

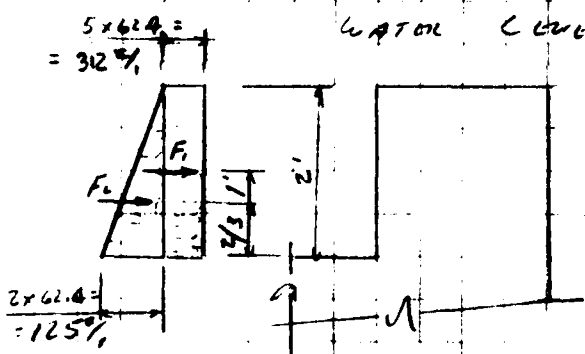
*** $v_c = .55 \times 2(f'c)^{1/2} = 60$ psi

$v_u = 21,600 \div 2' \times 7' \times 144 = 10.7$ psi < 60 psi OK

∴ No Reinf. Req'd.

Vert. Steel in End Sill

Water Level 5' Above Top of End Sill



$F_1 = 312 \times 2 = 624$ #/'
$F_2 = 125 \times 2 \div 2 = 125$ #/'
749 #/'

Mom = .624×1 + .125×2/3 = .707 k'/'

d = 24 - 4 - ½ = 19.5"

As = .707×12 ÷ 20 × .891 × 19.5 = .024 □"/'
#5 @ 18" (As = .21 □"/')

See SH 23, Temp Reinf Governs
∴ Use #5 @ 6"

Shear Obviously OK

CHECK TEMP & SHRINKAGE REINF

REF EM 1110-2-2103 Pg 5 PARA 13b(4)

As/FACE = .004 × 6 × 7.5 × 144 ÷ 2 = 4.32 $^{□"}$/FACE

4.32 ÷ 7.5 = .58 $^{□"}$/' USE #5@6 (As = .62 $^{□"}$/')

TEMP REINF PARALLEL TO RESTRAINED EDGE
∴ ALSO @ .40% ✓

∴ USE #5@4" IN VERTICAL DIRECTION

CHECK

As/FACE = $\frac{.004}{2}$ × 24 × 12 = .58 $^{□"}$/'

HNTB

MADE BY ____ DATE ____ JOB NO. ____
CHECKED BY JKB DATE 4/18/75 SEC. NO. ____
SHEET NO. ?-29

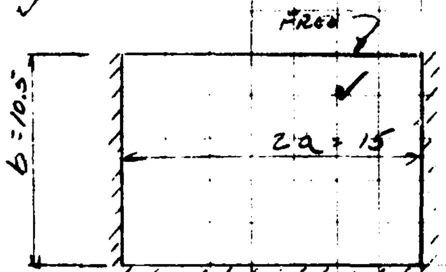
CALCULATIONS FOR Cox Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

End Sill (Upstream)

Case a & b as noted on Sh. No. 9 (H, N, T & B) will be investigated for max. mom.

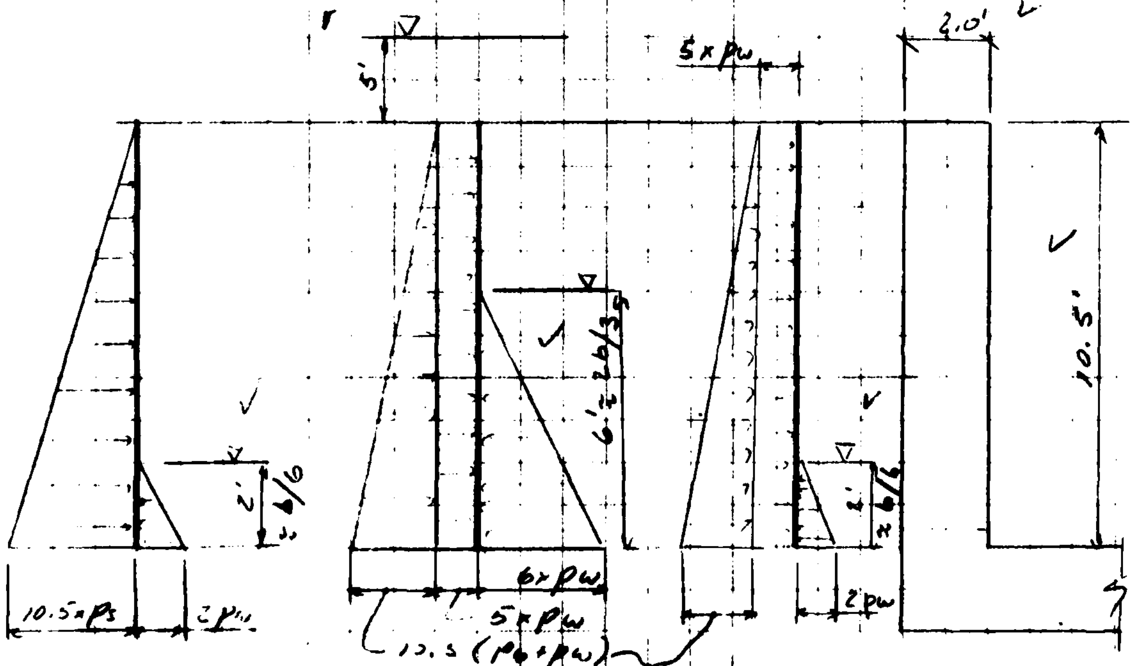
Ref. Eng. Monograph No. 27
"Moments & Reactions for Rect. Plates"
US Dept of Int.
Bureau of Reclamation



$a = 7.5$
$b = 10.5$

$a/b = 7.5/10.5 = .71$

Use 3/4 for a/b



Case a   Case b   Case b'

HNTB

MADE BY C.D. DATE 9/10/77 JOB NO. 9104

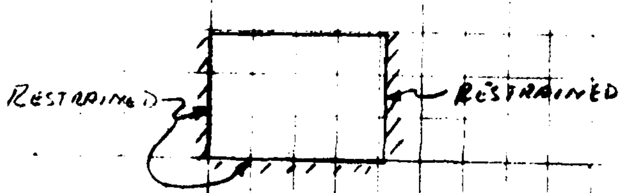
CHECKED BY JKT DATE 4 18.78 SEC. NO.

SHEET NO. 5-25

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

FIND MIN HORIZ & VERT REINF REQ DUE TO TEMP & SHRINKAGE

REF CIM 1110-2-2103 PG 5 5.(a)



USE MEMBER RESTRAINED ON OPPOSITE EDGES FOR BOTH HORIZ & VERT STEEL IN BOTH FACES. (CONSERVATIVE)

A_s/FACE $= \frac{.004}{2} \times 24" \times 12" = 0.58$ in²/'

∴ #7 @ 12" MIN (A_s = .60 in²/')

SOLVE FOR MAX MOM THAT WALL CAN TAKE WITH MIN. REINF.

$d = 20 - 4" - 1" = 15"$

$Mom = A_s \times f_s \times j \times d$

$M = .60 \times 20 \times .891 \times 15 \div 12 = 16.9$ k'/'

∴ DESIGN FOR MOM > 16.9 k'/'

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

HNTB

MADE BY C.O DATE [illegible] JOB NO. [illegible]
CHECKED BY JKJ DATE 4.18.75 SEC. NO.
SHEET NO. 5-26

CALCULATIONS FOR Cox Geo. [illegible]

Ps = 66.5 #/□
Pc = 35.3 #/□
Pw = 68.9 #/□

See H.N.T.B. [illegible]

CASE 4

(1) FIND MAX MOM. (POS & NEG) FOR VERT STEEL
(2) NEGLECT WATER IN BOX SINCE IT WILL REDUCE MOMENTS (MORE CONSERVATIVE)

FIG #

		+Mx	-Mx
#4 @ [illegible] 4.4/1	$10.5^2 \times .0665 \times 10.5 \times (.0584)$ $\times (-.0143)$	4.50 ᵏ/₁	—
		—	-1.10 ᵏ/₁
		+4.50 ᵏ/₁	-1.10 ᵏ/₁

BOTH MOMENTS LESS THAN 16.7 ᵏ/₁

∴ TEMP & SH REINF. [illegible]

(1) FIND MAX MOM. (POS & NEG) FOR VERT STEEL
(2) NEGLECT WATER IN BOX SINCE IT WILL REDUCE MOMENTS (MORE CONSERVATIVE)

FIG #

		+Mx	-Mx
#4 @ 8/0 @ 1/1	$10.5^2 \times .0665 \times 10.5 \times (.0433)$ $(-.0214)$	3.33 ᵏ/₁	—
		—	-1.65
		+3.33 ᵏ/₁	-1.65 ᵏ/₁

BOTH MOMENTS LESS THAN 16.5 ᵏ/₁
∴ TEMP [illegible] REINF. [illegible]

HNTB CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

MADE BY C.S. DATE 4/11/75 JOB NO. 9109
CHECKED BY J.K.J. DATE 4.18.75 SEC. NO.
SHEET NO. 5-22

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Case 6

(1) Find Max Mom. (Pos & Neg) for Vert. Steel.
(2) Neglect 6 x P.4 Load since it will reduce Mom. (Conservative)

Fig #		$+M_Y$	$-M_Y$
#1 @ 0/1	$5 \times .0624 \times 10.5^2 \times (.1212)$	4.17	—
#1 @ .6/1	$\times (-.0245)$	—	-0.84
#4 @ 0/1	$10.5^3 \times 0.0977 \times (.0584)$	6.61	—
#4 @ .6/1	$\times (-.0139)$	—	-1.57
		$10.78^{k'}/_1$	$2.41^{k'}/_1$

Both Moments Less than 16.5 k'/1
∴ Temp & Sh. Reinf. Good

(1) Find Max Mom (Pos & Neg) for Vert Steel
(2) Neglect 6 x P.4 Load since it will reduce Mom

Fig #		$+M_X$	$-M_X$
#1 @ 1/0	$5 \times .0624 \times 10.5^2 \times (.1788)$	6.15	—
#1 @ 1/1	$\times (.0807)$	—	-2.78
#4 @ 1/0	$10.5^3 \times 0.0977 \times (.0406)$	4.59	—
#4 @ 1/1	$(.0214)$	—	-2.42
		$10.74^{k'}/_1$	$-5.20^{k'}/_1$

Both Mom Less than 16.5 k'/1
∴ Temp & Sh. Reinf. Good.

Case 6'

Since water in Box was left neglected in Case 6, Results for Case 6' will be same as Case 6

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HNTB

MADE BY C.J. DATE 4/14/75 JOB NO. 9109
CHECKED BY JKA DATE 4/18/75 SEC. NO.
SHEET NO. 5-29

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

From Max Shear

Case 6 will obviously gov. (Neglect Expan Load)

Fig. #1 @ 0/1 .6745 x 5 + .0624 x 10.5 = 2.20 K/1

#4 @ 0/1 .4055 x 10.5² x .0377 = 4.37 K/1
6.57 K/1

See Sh 15 (HNTB) for Methods & Ref.

v_c = 60 psi

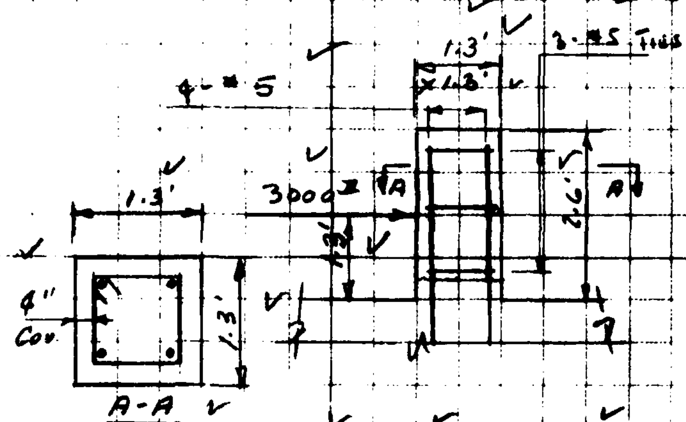
v_u = .104 V_u ÷ 4 = .104 x 6.570 ÷ 24
= 28.5 psi < 60 psi
OK

Since Reinf is Gov By the Temp & Shrinkage Requirements it will be ok to splice bar at same location.

Splice length #7 = 17.5* x 1.7** = 29.75" Say 30"

* Sh. No. 12 (HNTB)
** ACI STD 318-71 Pg 18 Class C Splice

Block Design



Dim. From Dwg D-1

3000# From Sh 11-3 By H.N.T.& B.

$M = 3000 \times 1.3 = 3,900'^{\#} = 3.9'^{k}$

Say $d = 1.3' \times 12 + 4'' - .62 - .31 = 10.67''$

$A_s = 3.9 \times 12 \div 20 \times .891 \times 10.67 = 0.25^{\square''}$

Use 4-#5 (1 each corner)
$A_s = .62^{\square''} > .25^{\square''}$ ok

Check Shear

$v_c = .55 \times 2(f'c)^{\frac{1}{2}} = 60$ psi (ACI working)

ACI 318-71 Pg 36 ¶ 11.2.1

$v_u = V \div b_w d = 3000 \div 1 \times 1.3 \times 12 \times 10.67$

$= 18.0$ psi < 60 psi ok

Use #5 @ 12" (Ties)

HNTB

MADE BY L.D DATE 4/10/75 JOB NO. 4204
CHECKED BY [illegible] DATE 4.18.75 SEC. NO.
SHEET NO. 5-30

CALCULATIONS FOR COY GLEN, ITHACA, N.Y.

CHECK UPLIFT

TRY SLAB 2'-6" THICK

SINCE SLAB IS RIGID ASSUME ENTIRE D.L. OF BOX IS SPREAD UNIFORMLY OVER CONTACT AREA BETWEEN SLAB & SOIL.

ALL COMPUTATIONS FOR UPLIFT BASED ON STRUCT. NO. 1.

USE AVE. DEPTH OF WATER IN DETERMINING BUOYANT FORCE.

POOL DEPTH OF 2' IN BOX (CONSERVATIVE)

HNTB

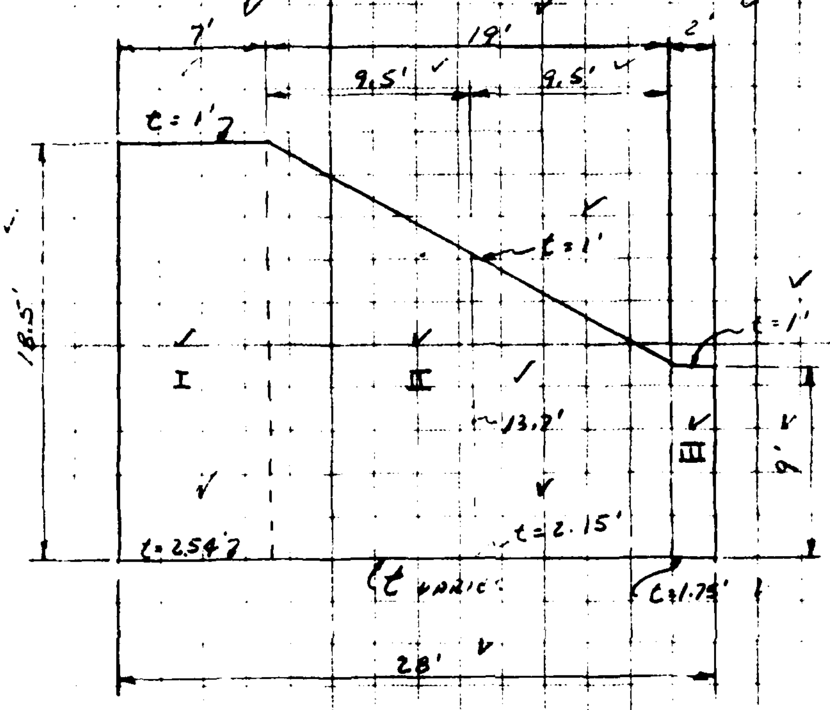
MADE BY L.D DATE 4/1?/75 JOB NO. 4204
CHECKED BY J.K.T DATE 4 18 75 SEC. NO.
SHEET NO. 5-31

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Box Walls

t = Thickness



$$Vol\ I = (1+2.54)\times\tfrac{1}{2}\times 18.5\times 7 = 229\ Ft^3$$

$$Vol\ II = \tfrac{19}{6}\left[(1+2.54)\times\tfrac{1}{2}\times 18.5 + 4(1+2.15)\times\tfrac{1}{2}\times 13.7 + (1+1.75)\times\tfrac{1}{2}\times 9\right] = 416\ Ft^3$$

$$Vol\ III = (1+1.75)\times\tfrac{1}{2}\times 2\times 9 = 25\ Ft^3$$

Σ 1 Wall = 670 Ft^3

Σ 2 Wall = 2 × 670 = 1340 Ft^3

End Sides

$$Vol_{up} = 2.0'\times 10.5'\times 15' = 315\ Ft^3$$

$$Vol_{Down} = (2+3)\times 2\times 15' = 150\ Ft^3$$

Σ End Sides = 465 Ft^3

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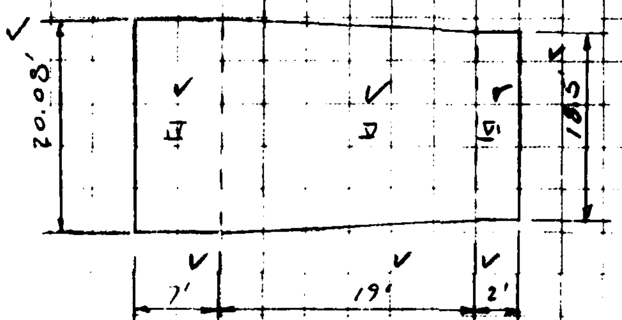
CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Blocks

$V_{BL} = 5 \times 1.3' \times 1.3' \times 2.6' = 22\ Ft^3$

Slab



Area IV = 7 × 20.08 = 140.6 Ft²
" V = (20.08 + 18.5) × ½ × 19 = 366.5 Ft²
" VI = 2 × 18.5 = 37.0 Ft²
544.1 Ft²

$V_{OL} = 544.1 \times 2.5 = 1360\ Ft^3$

Water (2' Deep)

Wt = .0624 × 2' × 15' × 24' = 45^k

Σ Conc Wt = .15 × (1340 + 465 + 22 + 1360) = 478^k

∴ Total Downward Load = 523^k

HNTB

MADE BY C.D. DATE 4/10/75 JOB NO. 4204
CHECKED BY J.L.? DATE 4 19 75 SEC. NO.
SHEET NO. 5-33

CALCULATIONS FOR COY GLEN, ITHACA, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

BUOYANT FORCE

FOR HEAD SEE SH 18 (H.N.T.&B.)

FTG THICKNESS

$h = 11.3' + 2.5' = 13.8'$

$P = 13.8 \times .0624 = .861$ K/□'

BUOYANT LOAD = 544.1 x .861 = 468 K

MIN. F.S. AGAINST UPLIFT = 523 ÷ 468 = 1.12

F.S. WILL BE GREATER THAN 1.12 SINCE FRICTION BETWEEN WALL AND EARTH, AND VERTICAL LOAD ON SLOPING OUTSIDE FACE OF BOX WALL HAS NOT BEEN INCLUDED.

BOTTOM SLAB DESIGN ✓

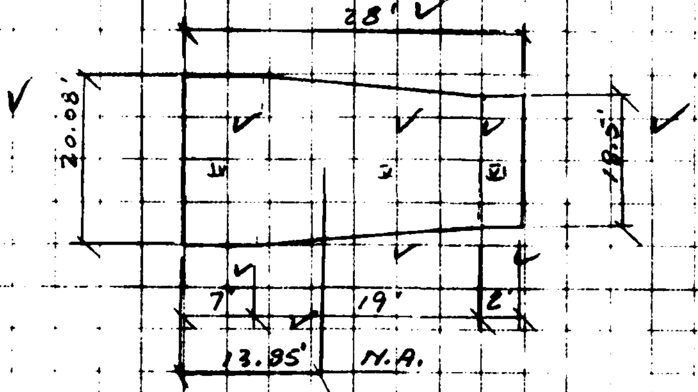
SINCE SLAB IS RIGID ASSUME D.L. OF BOX WILL RESULT IN A UNIFORM UPWARD EARTH PRESSURE. ✓

✓ LOADING CASE I ~ BOX EMPTY ✓ NO HYD. IMP. OR BUOYANT FORCES. (NEG. SLAB MOM.)

✓ LOADING CASE II ~ HYD. IMP. ✓ NO BUOYANT FORCES. (POS SLAB MOM.)

✓ SLAB THICKNESS AND HEIGHT OF DOOL WILL BE SELF CANCELLING IN THE DESIGN OF SLAB. ✓

Find Sect. Mom of Sec 3



	Area	d	Ad	$\bar{x}$	$A\bar{x}^2$		I_{oo}
IV	140.6	3.5	492	10.35	15,061		574
V	366.5	16.5	6047	2.65	2,574	*	11,020
VI	37.0	27.0	999	13.15	6,398		12
Σ	544.1		7538		24,033		11,606 = 35,639 ft

$\bar{x} = 7538 \div 544.1 = 13.85'$

* AISC 6th Ed. 1963 Pg 6-25

$C = \frac{19}{3}(2 \times 20.08 + 18.5) \div (20.08 + 18.5) = 9.63'$

$I_{oo} = 19^3(20.08^2 + 4 \times 20.08 \times 18.5 + 18.5^2) \div 36(20.08 + 18.5)$

$= 11,020$

S.M upstream $= 35,639 \div 13.85 = 2573$ ft^3

S.M downstream $= 35,639 \div 14.15 = 2519$ ft^3

HNTB

MADE BY [illegible] DATE 1/1/75 JOB NO. [illegible]
CHECKED BY JRT DATE 4.19.75 SEC. NO.
SHEET NO. S-36

CALCULATIONS FOR [illegible]

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Loading Case I (see Sh No 34)

[illegible]

= 573 - .15 × 1363 = 374^k (see Sh No 33)

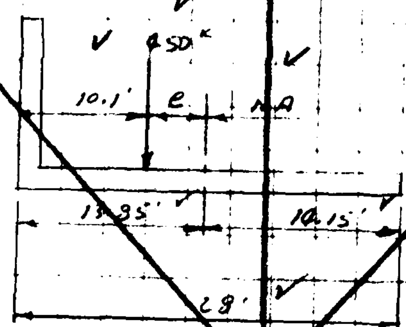
Uniform [illegible] Pres. Due to D.L. = 274^k ÷ 544.1^{ft} = .504 k/ft

LOADING CASE II (see Sh No 34)

[illegible]

= 6.0^k × 5 × 15 = 450^k

(Sh No A-3 [illegible])



e = 13.85 - 10.1 = 3.75'

M = 450 × 3.75 = 1688^{ik}

[illegible] $P = \frac{450}{544} + \frac{1688}{2519} = 1.48$ k/ft

[illegible] $P = \frac{450}{544} - \frac{1688}{2519} = 0.16$ k/ft

Net Soil Pressure

[illegible]

Total [illegible] = 523 - 468 = 55^k

P = 55 ÷ 544 = .101 k/ft

Deletion

42

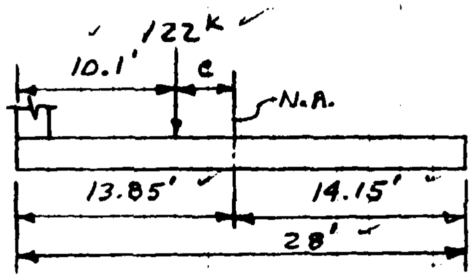
Page ___ of ___ pages.
Sheet 36A

Subject Coy Glen
Computation of
Computed by AJA Checked by RJG Date 1/12/76

LOADING CASE II

Total Downward Loading From Impact =

$1.63 \times 5 \times 15 = 122.25^{K}$
℄ see P. 12BA



$e = 13.85 - 10.1 = 3.75'$

$M = 122^{K} \times 3.75' = 458^{K'}$

Upstream $p = \frac{122}{544} + \frac{458}{2573} = .402\ ^{K}/S.F.$

Downstream $p = \frac{122}{544} - \frac{458}{2573} = .046\ ^{K}/S.F.$

Find Slab Load with Buoyant Condition and 2' pool of water in box.

Total Downward Load $\underset{P.38}{523^{K}} - \underset{P.39}{468^{K}} = 55^{K}$

$p = 55 \div 544 = .101\ K/SF \downarrow$

HNTB

CALCULATIONS FOR

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

MADE BY C.D. DATE 4/14/75 JOB NO. 4204
CHECKED BY [illegible] DATE 4.19.75 SEC. NO.
SHEET NO. 5-37

FIND DOWNWARD LOAD DUE TO 2.5' SLAB + 2' OF WATER

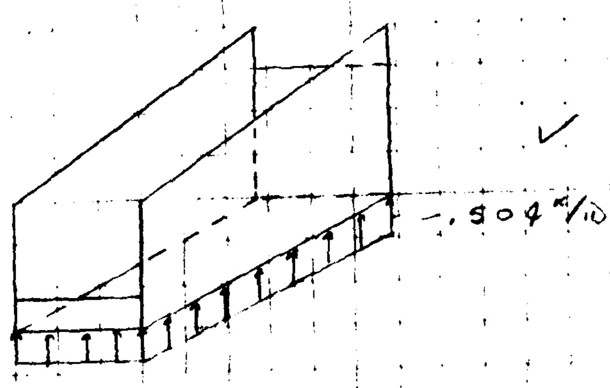
2 x .0624 = .125 k
2.5' x .15 k = .375 k
.500 k/lf

∴ DESIGN LOAD FOR THIS SITUATION

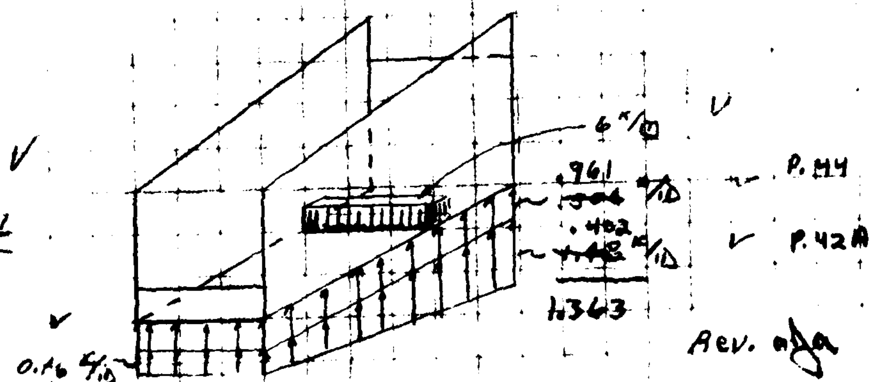
$-.101\ k/lf + .500\ k/lf = 0.399\ k/lf$

BY INSPECTION LOADING CONDITION WILL GOVERN FOR BOTTOM STEEL.

LOADING CASE I



LOADING CASE II



Rev. [illegible]

43

HNTB

MADE BY C.D. DATE 5/5/75 JOB NO. 4104
CHECKED BY JKJ DATE 5.8.75 SEC. NO.
SHEET NO. 5-37A

CALCULATIONS FOR Coy Glen, Ithaca N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Find Max Soil Pressure Under Box

Max. Soil Pressure will occur with 2' Pool of water in Box, Impact Condition and a Non-Buoyant Condition.

See Sheet 32

D.L. of Box + 2' Water = $523^{K} \div 544^{\square'} = .961\ K/\square'$

See Sh. 38

Imp Condition, Upstream Gate = ~~.504 + 1.48 = 1.984 K/□'~~

~~2.445 K/□'~~

P. 42A Rev. afa

.402

1.363 KS

Allowable bearing: 2 Tons/SF afa

Factor of Safety: $\frac{4.0}{1.363} = 2.9$ afa

HNTB

MADE BY C.D. DATE 9/14/75 JOB NO. 4204
CHECKED BY JRJ DATE 4.1975 SEC. NO.
SHEET NO. 5-38

CALCULATIONS FOR COY GLEN, ITHACA, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

TO DETAIL DESIGN MOM. IN ABUT SLABS USE "REINFORCED CONCRETE DESIGNER'S HANDBOOK" BY CHAS. E. REYNOLDS 6TH EDITION.

SEE HNTB SH NO 38A TO 38C

(REF. PG 206 & 207, FOR UNIFORM LOAD
(REF PG 214 & 215, FOR STRIP LOAD OF 6 K/FT

FOR THE TRIANGULAR LOAD IN LOADING CASE II, ASSUME AN AVERAGE UNIFORM PRESSURE.

LOADING CASE I (SEE SH NO 37)

CORNERS HELD DOWN

TABLE 38 { $R = 24' \div 15' = 1.6$

$K_B = 0.63 \quad K_L = 0.09$

(TRANS MOM) $M_B = K_B w L_B^2 \div 8 =$

$= -.63 \times (.504) \times 15^2 \div 8 = -8.9'^k/_1$

(LONG MOM) $M_L = K_L w L_L^2 \div 8 =$

$= -0.09 \times (.504) \times 24^2 \div 8 = -3.3'^k/_1$

HNTB

MADE BY ______ DATE ______ JOB NO. 9204

CHECKED BY ______ DATE ______ SEC. NO. ______

SHEET NO. 5-38A

CALCULATIONS FOR Coy Glen, Ithaca, NY

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

REINFORCED CONCRETE DESIGNER'S HANDBOOK

BY

CHAS. E. REYNOLDS

B.Sc.(Eng.), M.Inst.C.E.

SIXTH EDITION

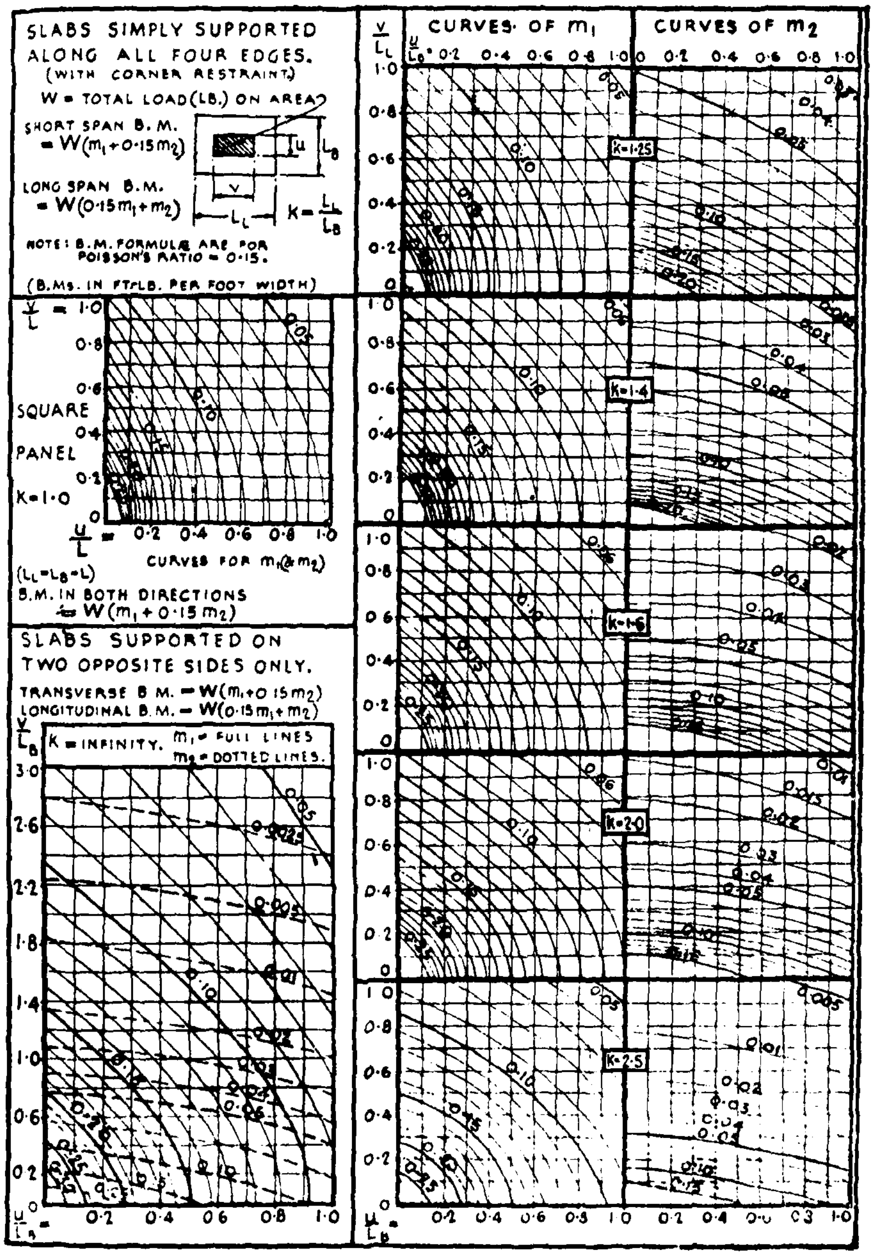
CONCRETE SERIES

PUBLISHED BY

CONCRETE PUBLICATIONS LIMITED

14 DARTMOUTH STREET, LONDON, S W 1

SLABS SPANNING IN TWO DIRECTIONS: RECTANGULAR PANELS. TABLE 42.
CONCENTRIC CONCENTRATED LOADS.



NOTE.—See note regarding square panels on page 214.

HNTB

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

MADE BY ________ DATE ________ JOB NO. 9204

CHECKED BY ________ DATE ________ SEC. NO. ________

SHEET NO. 5-38C

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

SLABS SPANNING IN TWO DIRECTIONS.

Notation.—The symbols used in the following and in *Tables* 38, 39, 41, 42 and 43 are as follows. (The corresponding symbols used in B.S. Code No. 114, where different, are given in brackets.) The symbols used in *Tables* 40 and 44 are given in the respective tables.

W = total load (lb.) on the slab and equal to wL_BL_L for a completely-loaded panel, and equal to wuv for a partially-loaded panel; w = uniformly-distributed load (lb. per sq. ft.).

$L_B(l_x)$ and $L_L(l_y)$ = short and long spans (ft.) respectively, $k = \frac{L_L}{L_B}$.

M_B and M_L = maximum bending moments at the midspan of the short and long spans respectively; M_{BA} and M_{BC} = bending moments at supports A and C respectively of the short span; M_{LD} and M_{LE}, the bending moments at supports D and E respectively of the long span. Bending moments are in ft.-lb. per foot width.

K_B and K_L = bending-moment reduction factors for short and long spans respectively, corners not held down; K_B' and K_L' = corresponding factors with corners held down.

$m_B(=\beta_x)$ and $m_L\left[=\beta_y\left(\frac{l_x}{l_y}\right)^2\right]$ = coefficients for positive bending moments on short and long snaps respectively, with corners held down; m_B' and m_L' = corresponding coefficients for negative bending moments. $m_{BO}(=\alpha_x)$ and $m_{LO}\left[=\alpha_y\left(\frac{l_x}{l_y}\right)^2\right]$ = coefficients for positive bending moments on short and long spans respectively with corners not held down.

Rectangular Panels Freely Supported along All Edges with Uniformly-distributed Load.—For a rectangular panel that is freely supported along all four edges in such a manner that the corners are free to lift, the Grashof and Rankine method is applicable and the bending moment reduction coefficients are $K_B = \frac{k^4}{k^4+1}$ and $K_L = 1 - K_B$. The midspan bending moments per foot width M_B and M_L are calculated from the formulæ in *Table* 38. The usual limit of application of this method is when the length of the panel is equal to twice the breadth, that is when $k = 2$. Beyond this limit the slab is considered to span across the short span only, the bending moment per foot width then being $\frac{wL_B^2}{8}$.

For the condition "corners not held down", the bending-moment coefficients in the B.S. Code correspond to m_{BO} and m_{LO} in *Table* 39.

In cases near the limit of $k = 2$, it is necessary to ensure that the amount of reinforcement in the long direction is not less than the minimum amount of distribution bars required.

For panels that are freely supported along all four edges but with the corners prevented from lifting, the corresponding coefficients K'_B and K'_L in *Table* 38 conform to a more exact analysis but with Poisson's ratio equal to zero.

The bending moments at midspan based on Dr. Marcus's method are the midspan bending moments calculated by the Grashof and Rankine method multiplied by a factor C; for a slab freely supported along all four edges $C = 1 - \frac{5}{6}\frac{k^2}{(1+k^4)}$; the midspan bending moments per foot width are $M_B = CK_B\frac{wL_B^2}{8}$ and $M_L = \frac{M_B}{k^2}$.

The resultant bending moments by the method of Dr. Marcus and the exact theory (with Poisson's ratio equal to zero) are almost identical. If Poisson's ratio is assumed to be 0·15, the midspan bending moments per foot of width are $M_B = \frac{wCK_BL_B^2}{8}\left(1 + \frac{0{\cdot}15}{k^2}\right)$ and $M_L = \frac{wCK_BL_B^2}{8}\left(0{\cdot}15 + \frac{1}{k^2}\right)$. Alternatively the appropriate coefficients can be obtained from the curves in *Table* 42 for $\frac{u}{L_B} = \frac{v}{L_L} = 1$ for a slab completely covered with a load of intensity $w = \frac{W}{L_BL_L}$. The bending-moment coefficients given in the B.S. Code for this case correspond to m_B and m_L in the top left-hand corner in *Table* 39.

Rectangular Panels Fixed along Four Sides with Uniformly-distributed Load.—If a panel is completely fixed along all four sides, the bending moments are as follows.

Short span: Midspan $M_B = +0{\cdot}8M_{BA}$; Support $M_{BA} = -K'_B\frac{wL_B^2}{8}$.

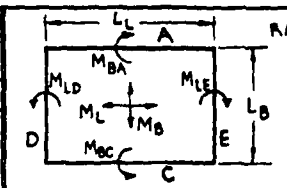
Long span: Midspan $M_L = +0{\cdot}8M_{LD}$; Support $M_{LD} = -K'_L\frac{wL_L^2}{8}$,

where K'_B and K'_L are as in *Table* 38. (See also B.S.-code method on page 208.)

(*Continued on page* 208.)

SLABS SPANNING IN TWO DIRECTIONS: RECTANGULAR PANELS. –TABLE 38.
UNIFORMLY-DISTRIBUTED LOAD.

RATIO OF SPANS	CONDITION ALONG FOUR EDGES				
	FREE CORNERS NOT HELD DOWN		FREE OR FIXED CORNERS HELD DOWN		FIXED (DR MARCUS)
k	K_B	K_L	K'_B	K'_L	C'
1·0	0·50	0·50	0·30	0·30	0·861
1·05	0·55	0·45	0·33	0·27	0·862
1·1	0·59	0·41	0·36	0·24	0·864
1·15	0·64	0·36	0·39	0·22	0·866
1·2	0·68	0·33	0·42	0·19	0·871
1·25	0·71	0·29	0·45	0·17	0·874
1·3	0·74	0·26	0·48	0·15	0·879
1·4	0·79	0·21	0·53	0·13	0·888
1·5	0·84	0·16	0·58	0·11	0·898
1·6	0·87	0·13	0·63	0·09	0·907
1·75	0·90	0·10	0·68	0·07	0·919
2·0	0·94	0·06	0·76	0·05	0·935
2·5	0·97	0·03	0·87	0·03	0·957
3·0	0·98	0·02	0·94	0·02	0·970



RATIO OF SPANS: $k = \frac{\text{LONG SPAN}}{\text{SHORT SPAN}} = \frac{L_L\,(\text{FT.})}{L_B\,(\text{FT.})}$

UNIFORMLY-DISTRIBUTED LOAD = w LB. PER SQ. FT.

FREELY-SUPPORTED ALONG FOUR EDGES.

CORNERS NOT HELD DOWN: $M_B = +K_B\frac{wL_B^2}{8}$; $M_L = +K_L\frac{wL_L^2}{8} = \frac{M_B}{k^2}$

CORNERS HELD DOWN: $M_B = +K'_B\frac{wL_B^2}{8}$; $M_L = +K'_L\frac{wL_L^2}{8}$

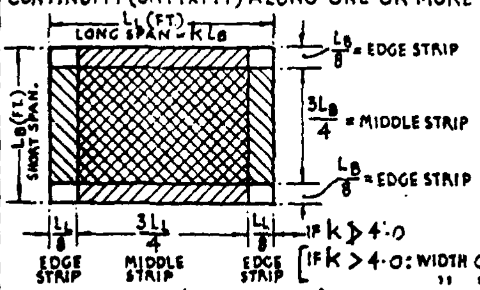
FIXITY ALONG FOUR EDGES.
CORNERS HELD DOWN. (DR MARCUS)

$M_B = +C'K_B\frac{wL_B^2}{24}$; $M_L = +C'K_L\frac{wL_L^2}{24} = \frac{M_B}{k^2}$

$M_{BA} = M_{BC} = -K_B\frac{wL_B^2}{12}$

$M_{LD} = M_{LE} = -K_L\frac{wL_L^2}{12} = \frac{M_{BA}}{k^2}$

CONTINUITY (OR FIXITY) ALONG ONE OR MORE EDGES. (B.S. CODE.)



[IF $k > 4\cdot0$: WIDTH OF MIDDLE STRIP = $L_L - L_B$
" " EDGE STRIPS = $0\cdot5\,L_B$]

CONDITIONS:
CORNERS HELD DOWN.
TORSIONAL RESISTANCE PROVIDED

NO REINFORCEMENT REQUIRED IN EDGE STRIPS TO RESIST BENDING MOMENT PARALLEL TO EDGES OF PANEL.

BENDING MOMENTS (FT.-LB. PER FOOT) IN MIDDLE STRIPS:–

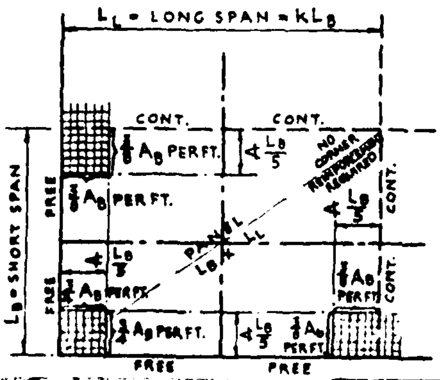
	AT MIDSPAN.	AT CONTINUOUS EDGE.	AT DISCONTINUOUS EDGE. (SLAB MONOLITHIC WITH SUPPORT)
SHORT SPAN:	$+m_B w L_B^2$	$-m'_B w L_B^2$	$-\frac{2}{3}m_B w L_B^2$
LONG SPAN:	$+m_L w L_L^2$	$-m'_L w L_L^2$	$-\frac{2}{3}m_L w L_L^2$

CORNER REINFORCEMENT FOR TORSIONAL RESISTANCE.
(B. S. CODE)

A_B & A_L = CROSS SECTIONAL AREA (PER FT.) OF REINFORCEMENT FOR POSITIVE B.M. AT MIDSPAN OF SHORT AND LONG SPANS RESPECTIVELY.

$\frac{3}{4}A_B$ & $\frac{3}{8}A_B$ = CROSS SECTIONAL AREA (PER FT.) OF CORNER REINFORCEMENT IN EACH OF TWO LAYERS (ONE NEAR TOP FACE OF SLAB; ONE NEAR BOTTOM FACE).

IF $A_L > A_B$ SUBSTITUTE $\frac{3}{4}A_L$ & $\frac{3}{8}A_L$ FOR $\frac{3}{4}A_B$ & $\frac{3}{8}A_B$



NOTE.– For values of m_B, m'_B, m_L and m'_L (for calculation of bending moments on middle strips) see *Table* 39.

SLABS SPANNING IN TWO DIRECTIONS (*continued*).

Square Panels ($k = 1{\cdot}0$).—The expression in *Table* 42 that the bending moment in both directions is $W(m_1 + 0{\cdot}15m_2)$ applies only if load is over entire panel, or if $u = v$.

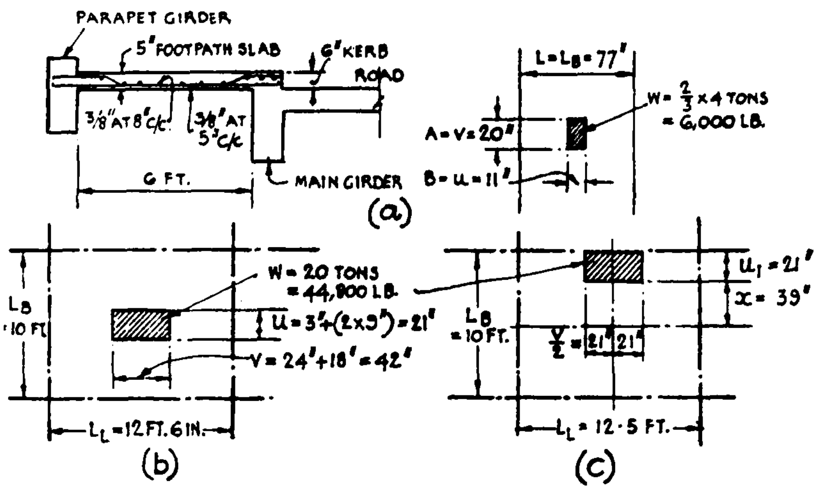
Other conditions.—u and v can be in either direction; m_1 is the bending moment coefficient in the direction of u; m_2 is the coefficient in the direction of v. Coefficient m_1 is based on $\frac{u}{L}$ and $\frac{v}{L}$ as selected; for coefficient m_2 reverse u and v.

Example. If $\frac{u}{L} = 0{\cdot}8$ and $\frac{v}{L} = 0{\cdot}2$, $m_1 = 0{\cdot}072$; for $\frac{u}{L} = 0{\cdot}2$ and $\frac{v}{L} = 0{\cdot}8$, $m_2 = 0{\cdot}103$.

Bending moments.

On span in direction of u: $W[0{\cdot}072 + (0{\cdot}15 \times 0{\cdot}103)] = 0{\cdot}087W$ ft.-lb. per ft.
On span in direction of v: $W[0{\cdot}103 + (0{\cdot}15 \times 0{\cdot}072)] = 0{\cdot}114W$ ft.-lb. per ft.

Examples of Panels Supporting Concentrated Loads.—The following examples illustrate the use of *Tables* 42 and 43 for slabs supporting a load which is concentrated uniformly over an area less than the entire area of the panel. Notes on these tables are given on page 32.



(*a*) The footpath of a bridge spans 6 ft. between a parapet girder and a main longitudinal girder, and is monolithic with both girders [diagram (*a*)]. The live load is either 100 lb. per sq. ft. uniformly distributed, or a load of 4 tons from a wheel the contact area of which is 12 in. by 3 in. (With the latter load the stresses may be increased by 50 per cent.; that is at ordinary working stresses the wheel load can be assumed to be about 6000 lb.) These loads comply with the recommendations of the Ministry of Transport.

(i) Assume a 5-in. slab; total uniformly-distributed load $= 63 + 100 = 163$ lb. per sq. ft. With continuity at both supports, bending moment at midspan and at each support is $\frac{1}{12} \times 163 \times 6{\cdot}3^2 \times 12 = 6400$ in.-lb. per ft. width.

(ii) Contact area of 12 in. by 3 in. at the wheel can be increased to 20 in. by 11 in. (*Table* 6); depth to the reinforcement is about 4 in.

The slab spans mainly in one direction; the curves in the lower left-hand corner of *Table* 42 apply. $\frac{u}{L_B} = \frac{11}{77} = 0{\cdot}143$; $\frac{v}{L_B} = \frac{20}{77} = 0{\cdot}26$; $m_1 = 0{\cdot}22$ and $m_2 = 0{\cdot}12$.

Free transverse bending moment $= 6000[0{\cdot}22 + (0{\cdot}15 \times 0{\cdot}12)]12 + (\frac{1}{8} \times 63 \times 6{\cdot}3^2 \times 12)$ $= 17{,}150 + 3750 = 20{,}900$ in.-lb. per ft. width.

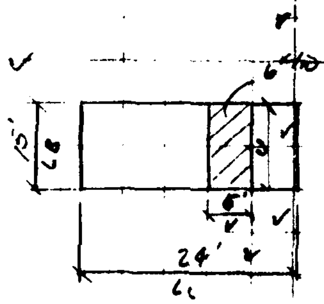
Allow for continuity (partial fixity) by reducing the free bending moment due to the dead load by one-third, and that due to the live load by 20 per cent.; the transverse bending

(*Continued on page 216.*)

LOADING CASE II (SEE SH NO 37) ✓

STRIP LOADING = $6^{K}/\square$ ✓

$W = 6 \times 15 \times 5 = 450^{K}$ ✓



$R = 24 \div 15 = 1.6$

$V \div C_L = 5 \div 24 = .208$

$U \div C_B = 15 \div 15 = 1.0$

FROM CHART ON TABLE 42 $R = 1.6$

$m_1 = .084$ ✓
$m_2 = .058$ ✓

(MOM TRANS) $M_B = W(m_1 + .15 m_2) =$
$= 450(.084 + .15 \times .058) = +41.7^{K'}/'$

(LONG MOM) $M_L = W(.15 m_1 + m_2) =$
$= 450(.15 \times .084 + .058) = +31.8^{K'}/'$

FOR TRIANGULAR LOAD

$P = (.16 + 1.48) \times \frac{1}{2} = 0.82^{K}/\square$

PROPORTION Load of .82 k/□ with Columns Case I of 0.504 k/□ (Sh No 39)

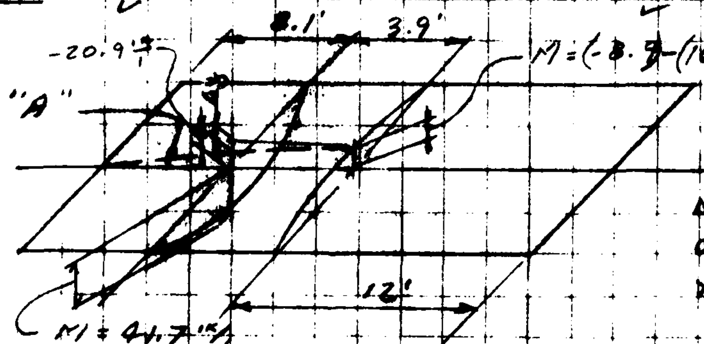
(TR MOM) M_{13} = 0.82 × 8.9 ÷ .504 = −14.5 'k/'

(LG MOM) M_C = 0.82 × 3.3 ÷ .504 = −5.4 'k/'

	M TRANS	M LONG
UNIFORM LD (.504 k/□)	−8.9 'k/'	−3.3 'k/'
TRIAG. LD (.16 TO 1.48 k/□)	−14.5 'k/'	−5.4 'k/'
Σ NEG MOM.	−23.4 'k/'	−8.7 'k/'

MAX MOM. FOR BOT SLAB CEILING CASE II

TRANS MOM



$\Delta y = a x^2$
$a = M = 23.4$ 'k
$x = 3.9/12$

SINCE POS & NEG MOM DO NOT OCCUR AT THE SAME LOCATION, CALCULATE NEG. MOM ASSUMING PARABOLIC CURVE "A" WITH ORDINATE AT ℄ OF 23.4 'k/'

$\therefore -M = -23.4 + \underbrace{\left(\frac{3.5}{12}\right)^2 \times 23.4}_{\Delta y} = -20.9$ 'k/'

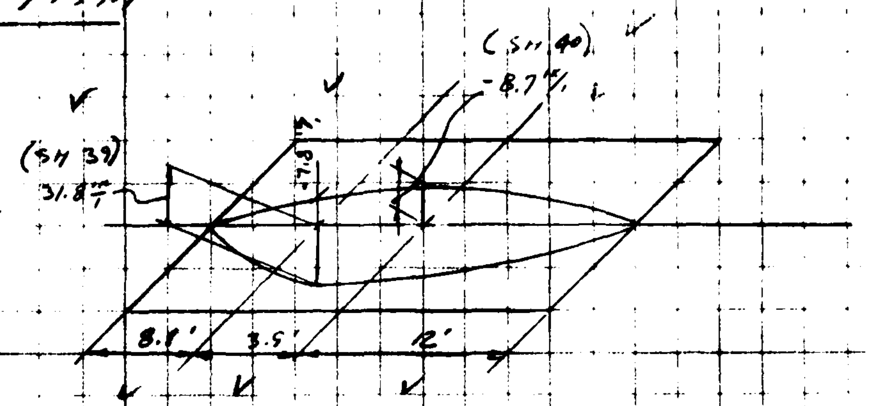
$\therefore \Sigma M = +41.7 - 20.9 = 20.8$ 'k/'

HNTB

MADE BY ___ DATE 4/16/75 JOB NO. 4202
CHECKED BY JRJ DATE 4/21/75 SEC. NO.
SHEET NO. S-41

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

Longit Mom



Find -Mom @ Point of Max Pos Mom

$\therefore -M = -8.7 + \left(\frac{3.5}{12}\right)^2 \times 8.7 = -7.8'k/'$

$\therefore$ Design Mom = 31.8 - 7.8 = 24.0'k/'

Max Mom and Top Steel Loading Case I

Trans Mom = -8.9'k/'

Long Mom = -3.3'k/'

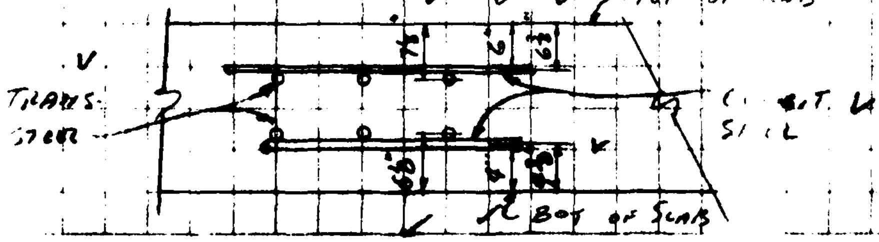
} See Sh. No. 33

Find d for Top & Bot Steel

Ref EM 1110-2-2103 1%

Cl Cov Top Steel = 6"

" " Bot " = 4"



HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

ASSUME #6 BAR

$d_{TOP\ LONG.} = 30" - 6" - .375" = 23.625"$ USE 23.5"

$d_{TOP\ TRAN} = 30" - 6" - .75" - .375" = 22.875"$ USE 22.5"

$d_{BOT\ TRANS} = 30" - 4" - .75" - .375" = 24.875"$ USE 24.5"

$d_{BOT\ LONG} = 30" - 4" - .375" = 25.625"$ USE 25.5"

TRANS BARS

TOP $A_s = 8.9 \times 12 \div 20 \times .891 \times 22.5 = .27^{□}/'$ < MIN (#6@ 12")

BOT $A_s = 20.8 \times 12 \div 20 \times .891 \times 24.5 = .58^{□}/'$ (#7@ 12")

LONGIT BARS

TOP $A_s = 3.3 \times 12 \div 20 \times .891 \times 23.5 = 0.10^{□}/'$ < MIN (#6@ 12")

BOT $A_s = 24.0 \times 12 \div 20 \times .891 \times 25.5 = 0.64^{□}/'$ (#5@ 12" / #6@ 12")

MIN REINFORCEMENT REF EM 1110-2-2103
PG 5 (UNRESTRAINED MEM)

MIN LONG & TRANS $= 30 \times 12 \times .0025 \div 2 =$

$= .45^{□}/'$

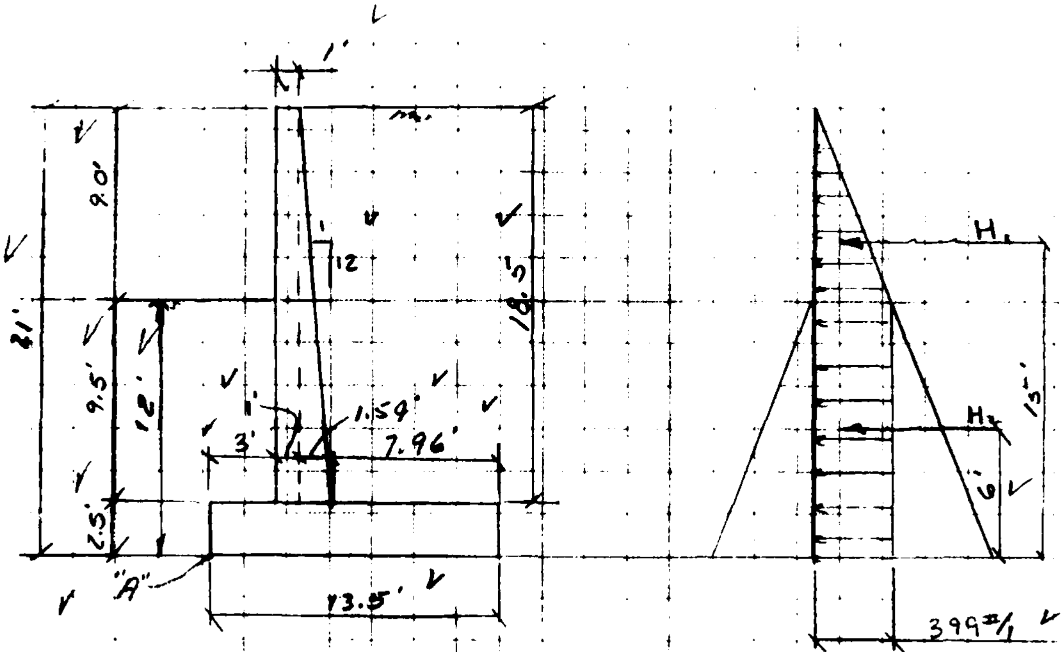
USE #6@ 12" (MAX) $A_s = .44^{□}/'$

HNTB

MADE BY ____ DATE ____ JOB NO. ____
CHECKED BY ____ DATE 4 21 75 SEC. NO. ____
SHEET NO. 5-43

CALCULATIONS FOR [illegible]

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



FOR CALCS ~ SATURATED COND

$\gamma_s = 133\ \#/ft^3$ } SEE SH. A1

$\rho_s = 49.3\ \#/ft^3$ } HNTB

				O.T.M.
$H_1 = 399 \times 9.0 \div 2$	$= 1795\#$	$\times 15'$	$=$	$26,925'\#$
$H_2 = 399 \times 12$	$= 4,788\#$	$\times 6'$	$=$	$28,728'\#$
ΣH	$= 6583\#$		OTM =	$55,653'\#$

HNTB

MADE BY ____ DATE ____ JOB NO. ____
CHECKED BY J.K.J. DATE 4.2.75 SEC. NO. ____
SHEET NO. 5-44

CALCULATIONS FOR [illegible]

[illegible]

			V		M
Conc	Ftg	.150 × 2.5 × 13.5 =	5.06	× 6.75 =	34.2 k
	Stem	.150 × 1.0 × 18.5 =	2.78 k	× 3.5 =	9.7 k
	Stem	.150 × 18.5 × 1.54/2 =	2.14 k	× 4.51 =	9.7
Σ Conc			9.98 k		53.6 k
Earth	Toe	.133 × 2.0 × 9.5 =	3.79 k	× 1.5'	5.7 k
	Back	.133 × 18.5 × 1.54/2 =	1.93 k	× 5.03'	9.5 k
	Heel	.133 × 18.5 × 7.96 =	19.59 k	× 9.52 =	186.5 k
Σ Conc + Σ Earth			35.25 k		255.3 k

R.M ÷ O.T.M = 255.3 ÷ 55.7 = 4.58 > 2.0 OK

ΣH ÷ ΣV = 6.58 ÷ 35.25 = 0.187 < .333 OK

ΣM @ Pt A = 255.3 − 55.7 = 199.6 'k

[illegible] 199.6 ÷ 35.25 = 5.66 [illegible]

e = 6.75' − 5.66' = 1.09'

S.M = $bh^2 \div 6 = 1 \times 13.5^2 \div 6 = 30.4$ Ft³

pressure = $\frac{V}{A} \pm \frac{Ve}{S.M} = \frac{35.25}{13.5} \pm \frac{35.25 \times 1.09}{30.4}$

= 2.61 ± 1.26 = 3.87 k/□ &
1.35 k/□

OK Less Than 4 k/□

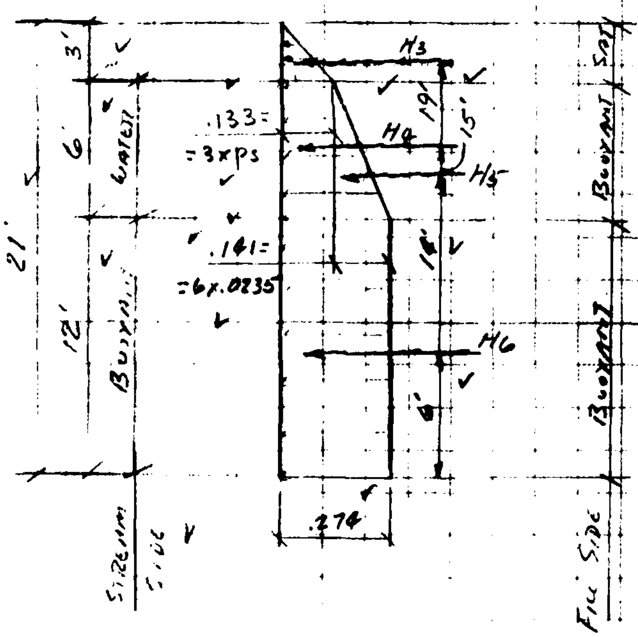
HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

HNTB

MADE BY L.D. DATE 4/ /75 JOB NO. 42 4
CHECKED BY JKT DATE 4.21.75 SEC. NO.
SHEET NO. 5-95

CALCULATIONS FOR

CHECK BUOYANT CONDITION



SATURATION*

$w_s = .133\ ^k/_{ft^3}$

$p_s = .0443\ ^k/_{ft^2}$

BUOYANT*

$w_B = .0706 + .0624 = .133\ ^k/_{ft^3}$

$p_B = .0235 + .0624 = .0859\ ^k/_{ft^2}$

* See SH. No. A-1 (HNTAR)

H_3	$.133 \times 3 \div 2$	=	$.20^k$	×	19'	=	$3.8^{'k}$
H_4	$.133 \times 6$	=	$.80^k$	×	15'	=	$12.0^{'k}$
H_5	$.141 \times 6 \div 2$	=	$.42^k$	×	14'	=	$5.9^{'k}$
H_6	$.274 \times 12$	=	3.29^k	×	6'	=	$19.7^{'k}$
		ΣH =	4.71^k			OTM =	$41.4^{'k}$

FOR R.M. ADD 6' OF WATER ON TOE TO THE RESULTS OF SATURATED SOIL CONDITION

		V		M
F Cond. & Check		35.25^k		$255.3^{'k}$
6' Water $3 \times 6 \times .0624$	=	1.12^k	× 1.5	$1.7^{'k}$
		36.37^k		$257.0^{'k}$
Buoy. $18 \times .0624 \times 13.5$	=	-15.19^k	× 6.75	-102.5
		21.18^k		$154.5^{'k}$

$R.M. \div OTM = 154.5 \div 41.4 = 3.73 > 2.0$ OK

$\Sigma H \div \Sigma V = 4.71 \div 21.18 = .222 < .333$ OK

HNTB

MADE BY C.J. DATE 9/1/75 JOB NO. 42
CHECKED BY J.K.J DATE 4 21 75 SEC. NO.
SHEET NO. 5-46

CALCULATIONS FOR

ΣM @ "A" = 154.5 − 41.4 = 113.1'k

Res. is 113.1 ÷ 21.18 = 5.34' from Pt A

e = 6.75 − 5.34 = 1.41'

pressure = $\frac{V}{A} \pm \frac{Ve}{S}$ = $\frac{21.18}{13.5} \pm \frac{21.18 \times 1.41}{30.4}$ =

= 1.57 ± 0.98 = 2.55 k/□ + 0.59 k/□

OK Less than 4 k/□

Stem Steel

(Sht 43) Mom @ Base = 1.80 × 16.5' + .395 × 9.5 ÷ 2 = 40.5 k'/1

V @ Base = 1.8 + .395 × 9.5 = 5.55 k/1

Mom 9.5' up = 1.80 × 3 = 5.4 '/1

V 9.5' up = 1.8 k/1

#8 Bars

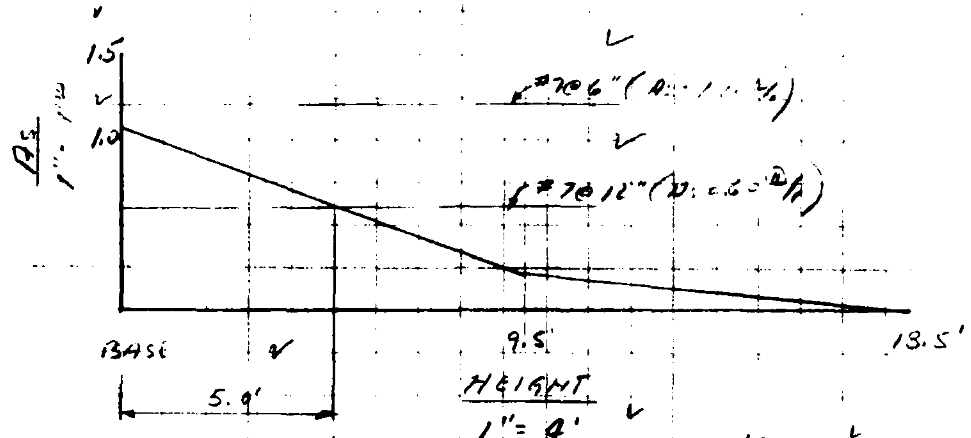
d = 2.50 × 12 − 4 − ½ = 25.5 Use 26"

= (12 + 9 ...) = 16.5" Use 16."

As base = 40.5 × 12 ÷ 20 × .891 × 26 = 1.05 □"/1

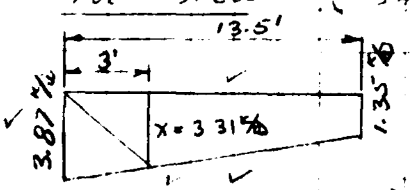
As 9.5' up = 5.4 × 12 ÷ 20 × .891 × 16 = 0.22 □"/1

58



d@ Theor. Cut-off = 12" + 13.5 - 9.5 = 21" ...
For Splice of Dowel use 1'7" (See Sh ... 26)

Toe Steel (... ...)



$x = 1.35 + \frac{10.5}{13.5} \times (3.87 - 1.35)$

$x = 3.31$ K/D

		V		M
	3.87 × 3 ÷ 2	= 5.81ᴷ	× 2	= 11.6ᴵᴷ
	3.31 × 3 ÷ 2	= 4.97ᴷ	× 1	= 5.0ᴵᴷ
Conc	2.5 × 3 × .15	= -1.13ᴷ	× 1.5	= -1.7ᴵᴷ
EA	9.5 × 3 × .13	= -3.79ᴷ	× 1.5	= -5.7
		5.86 K/l		9.2 IK/l

d = 30 - 4 - ½ = 25.5"

As = 9.2 × 12 ÷ 20 × .891 × ... = .25 □/l

Bars #7@12" (As .60 □/l) From Stem into Ftg.

HNTB

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

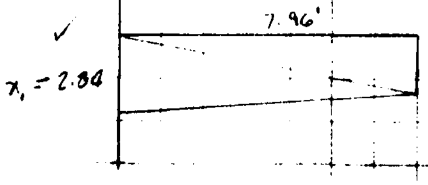
CALCULATIONS FOR

MADE BY ____ DATE ____ JOB NO. ____
CHECKED BY 7 A 7 DATE 1 2 2 75 SEC. NO. ____
SHEET NO. 5- 48

Mom. ... (... Cant...)

$x_1 = 1.35 + \frac{7.96}{13.5}(3.87 - 1.35)$

$x_1 = 2.84$



		V		
	$1.35 \times 7.96 \div 2 =$	5.31	$\times 5.31 =$	28.5
	$2.84 \times 7.96 \div 2 =$	11.30	$\times 2.65 =$	30.0
1.35	$2.5 \times 7.96 \times .15 =$	−2.99	$\times 3.98 =$	−11.9
	$18.5 \times 7.96 \times .133 =$	−19.59	$\times 3.98 =$	−78.0
		−5.91		−31.4

$A_s = 31.4 \times 12 \div 20 \times .891 \times 45.5 = 0.83 ^{\square''}/_{'}$

Use #6 @ 6" ($A_s = 0.88 ^{\square''}/_{'}$)

Temp & Shrinkage Reinf. Ref. EM 1110-2-2103

For ... use ... Restrained ... Pg 5 b(2)

Avg. ... A_s/face $= \frac{.004}{2} \times (1.0 + 2.54) \times \frac{12}{2} \times 12 = .51 ^{\square''}/_{'}$

(see Sh 63) Restrained ... = 18 − 2.5 − 5 = 10.5'

∴ 10.5 ÷ 4 = 2.6' Say 3'

For lower 3' use #7 @ 12" (...)

For Remainder of ... Pg 5 b(1)

A_s/face $= \frac{.0025}{2}(1.0 + 2.54) \times \frac{12}{2} \times 12 = .32 ^{\square''}/_{'}$

For Remainder ... use #5 @ 12" (A_s .31) (...)

Also use #5 @ 12" for ... Main Face (...) (A_s .31 $^{\square''}/_{'}$)

40

HNTB

MADE BY ___ DATE ___ JOB NO. ___
CHECKED BY JKJ DATE 3 22 75 SEC. NO. ___
SHEET NO. 5-41

CALCULATIONS FOR ___ Geen, Ithaca ___

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

(See Manor Building)

(See SH No 6)

Ave. Vol Conc. Cant ≅ $(12 + 18.25) \times 2 \times 6.25 \times \frac{1}{12} \times 5 =$

$= 39.4$ ft³

$V = .15 \times 39.4 = 5.91^k$

$M = 5.91 \times 2.1 = 10.8'^k$

$d = 7.5 \times 12 - 4 + 2 = 55.5''$

$A_s = 10.8 \times 12 \div 20 \times .891 \times 55.5 = .12^{\square}$ < $.12^{\square}/ft$

2 - #5 Supplied $A_s = .62^{\square}$

(See Sh. No 22) $v_c = .55 \times 2 \times (f'_c)^{\frac{1}{2}} = 60$ psi

$v_u = 5{,}910 \div (39.25) \times 6.25 \times 12 = 5.2$ psi

Stirrups not Req'd

Use #5 @ 12"

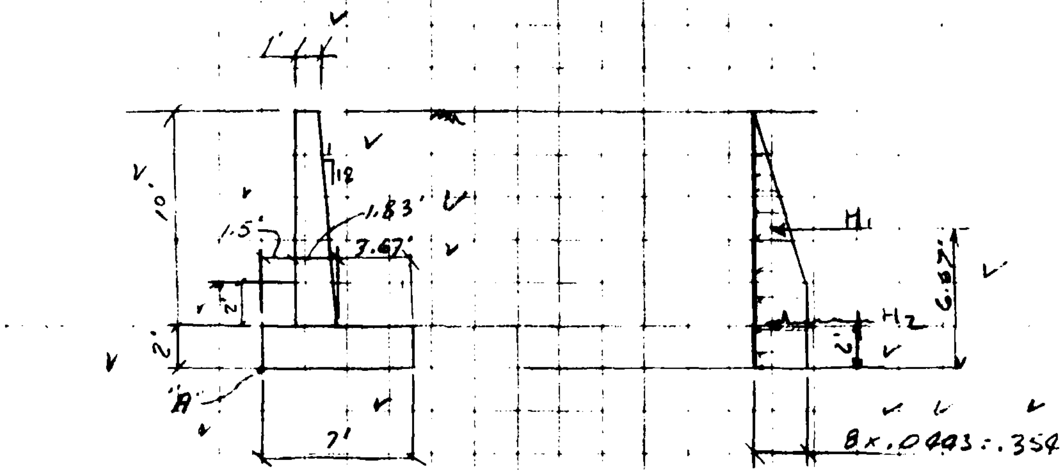
HNTB

MADE BY ____ DATE 9/16/?? JOB NO. 41??
CHECKED BY J.K.T. DATE 4.22.75 SEC. NO.
SHEET NO. 5-50

CALCULATIONS FOR ... Glen, ... Branch ...

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Retaining Wall



w_s = 133 #/ft³ } SH ...
p_s = 44.3 #/ft² } HNTB

H_1 = 0.354 x 8 ÷ 2	= 1.42^K	x 6.67'	= 7.9^{'K}
H_2 = 0.354 x 4	= 1.42^K	x 2.0'	= 2.8^{'K}
	2.84^K		12.2^{'K}

Turn. Mom. about "A"

		V		M
Conc Ftg	.150 x 7 x 2	= 2.10^K	x 3.5'	= 7.4^{'K}
Stem	.150 x 1 x 10	= 1.50^K	x 2.0'	= 3.0^{'K}
Stem	.150 x 10 x .83/2	= 0.62^K	x 2.79'	= 1.7^{'K}
Σ Conc		4.22^K		12.1'
Earth Toe	.133 x 2 x 1.5	0.40^K	0.75'	0.3^{'K}
Br	.133 x 10 x .83/2	= 0.55^K	3.05'	1.7^{'K}
	.133 x 10 x 3.67	= 4.88^K	5.17'	= [illegible]
Σ Conc + Earth		10.05^K		39.3^{'K}

RM = 39.3 ... 12.2 ...

... : ΣH / ΣV = 2.84 ÷ 10.05 = .282 < .333 ...

HNTB

MADE BY C.D DATE 9/16/75 JOB NO. 9204
CHECKED BY J.K.T DATE 9.22.75 SEC. NO.
SHEET NO. 5-51

CALCULATIONS FOR Cox Cell, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

ΣM @ Pt "A" : 39.8 - 12.7 = 27.1'k

Result is 27.1 ÷ 10.05 = 2.70' from Pt "A"

e = 3.5 - 2.7 = 0.8'

S.M. $bh^2 \div 6$ = 1 × 7^2 ÷ 6 = 8.17 ft³

pressure = $\frac{10.05}{7} \pm \frac{.8 \times 10.05}{8.17}$ = 1.44 ± .98

= 2.42 k/□

= 0.46 k/□

OK < 4 k/□

Bouyant Condition will not govern
(See Calc. for 18.5' High Stem)

Stem Steel

Sat. Cond. will Gov.

M_{stem} @ Base = 1.42 × 4.67 + .354 × 2 × 1 = 7.3'k/'

V @ Base = 1.42 + .354 × 2 = 2.13k/'

M_{stem} @ ... = 1.42 × 2.67 = 3.8'k/'

V @ ... = 1.42k/'

d @ Base = 1.83 × 12 - 4 - ½ = 17.46 say 17.5"

A_s = 7.3 × 12 ÷ 20 × .891 × 17.5 = .28 □"/'

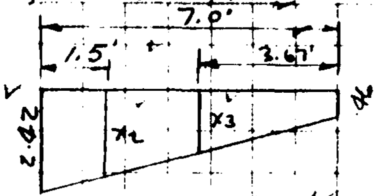
Use #5 @ 12" Full Height A_s = .31 □"/'

HNTB

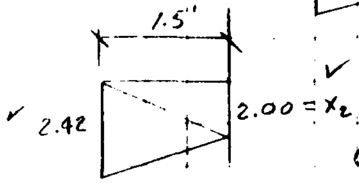
MADE BY ___ DATE ___ JOB NO. ___
CHECKED BY DKT DATE 4.22.75 SEC. NO. ___
SHEET NO. 5-32

CALCULATIONS FOR Box Culv, Ithaca, N.Y.

Toe Steel



$X_2 = .46 + \frac{5.5}{7}(2.42-.46)$
$X_2 = 2.00$



		V		M
	2.42 × 1.5 ÷ 2 =	1.82	× 1.0 =	1.8
	2.00 × 1.5 ÷ 2	1.50	.5 =	0.8
Conc	.15 × 2 × 1.5	− 0.45	.75 =	− 0.3
Earth	.133 × 2 × 1.5	− 0.40	.75 =	− 0.3
		2.47 k/		2.0 k/

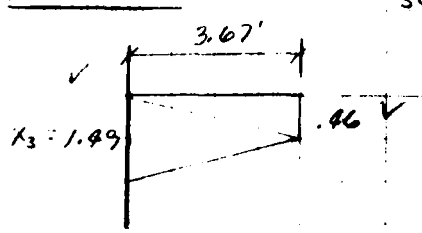
d = 24 − 4 − .5 = 19.5

As = 2 × 12 ÷ 20 × .885 × 19.5 = 0.07 □"/'

Use #5 @ 12" (Bend from Stem)

Heel Steel

See sketch above $X_3 = .46 + \frac{3.67}{7}(2.42 - .46)$
$X_3 = 1.49$



		V		M
.46 × 3.67 ÷ 2 =		0.84	× 2.45 =	2.1
1.49 × 3.67 ÷ 2 =		2.73	× 1.22 =	3.3
.15 × 3.67 × 2		− 1.10	× 1.84	− 2.0
.133 × 3.67 × 10		− 4.88	× 1.84	− 9.0
		− 2.41 k/'		− 5.6 k/'

As = 5.6 × 12 ÷ 20 × .885 × 19.5 = .19 □"/'

Use #5 @ 12" (As = .31 □"/')

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

MADE BY L.D. DATE 4/17/75 JOB NO. 7607
CHECKED BY DKJ DATE 4/22/75 SEC. NO.
SHEET NO. 5-53

HNTB

CALCULATIONS FOR Cay. Glen, Ithaca, N.Y.

Temp + Shrinkage Steel

Ref EM 1110-2-2103 Pg 5 b(3)

Mem. Restrained @ one edge

Ave. A_s/Face $= \frac{.004}{2} \times (1+1.83) \times \frac{12}{2} \times 12 = .41^{\square}/_1$

Use #6 @ 12" (A_s = .44$^{\square}$/1)

See 5-62 } Restrained edge ℄ 18' - 2.5' = 15.5'

∴ 15.5 ÷ 4 = 3.6'

For Crack 3' use #6 @ 12

For remainder of Horiz Steel Pg 5 b(1)

A_s/Face $= \frac{.0025}{2} (1+1.83) \times \frac{12}{2} \times 12 = .25^{\square}/_1$

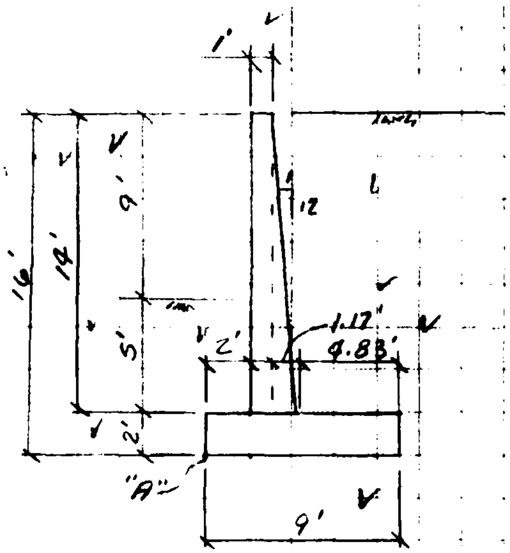
Use #5 @ 12" (A_s = 0.31$^{\square}$/1)

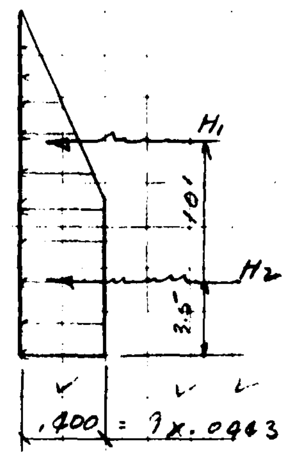
HNTB

MADE BY CD DATE 4/15/75 JOB NO. 9100
CHECKED BY JKJ DATE 4 22 75 SEC. NO.
SHEET NO. 5-30

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

Retaining Wall





As noted from calc. of High Wall, the governing cond. will be saturated soil.

γ_s 133 #/ft³ } See Sh. A1
p_s 44.3 #/ft² } H.N.T.C.B.

$H_1 = .4 \times 9 \div 2 = 1.8^k \times 10 = 18.0'^k$

$H_2 = .4 \times 7 = 2.8^k \times 3.5 = 9.8'^k$

$\Sigma H = 4.6^k$ O.T.M. $= 27.8'^k$

HNTB

MADE BY L D DATE 4/12/75 JOB NO. 9264
CHECKED BY JKJ DATE 4.22.75 SEC. NO.
SHEET NO. 5-55

CALCULATIONS FOR Cox Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Take Mom. About Pt "A"

			V			M
Conc.	Ftg	.150 × 2 × 9 =	2.70k ×	4.5	=	12.2 'k
	Stem	.150 × 1 × 14 =	2.10k ×	2.5	=	5.3
	Stem	.150 × 14 × 1.17 ÷ 2 =	1.23k ×	3.39	=	4.2 'k
Σ Conc		=	6.03k/l			21.7 'k/l
Earth	Toe	.133 × 2 × 5 =	1.33k ×	1.0'	=	1.3 'k
	Back	.133 × 14 × 1.17 ÷ 2 =	1.09k ×	3.78'	=	4.1 'k
	Heel	.133 × 14 × 4.83 =	8.99k ×	6.59'	=	59.3 'k
Σ Conc. + Earth			17.44k/l			86.4 'k/l

(Sh 54)

R.M ÷ O.T.M = 86.4 ÷ 27.8 = 3.11 > 2.0 ok

ΣH ÷ ΣV = 4.6 ÷ 17.44 = .264 < .333 ok

ΣM on about Pt "A" = 86.4 − 27.8 = 58.6 'k/l

Result is 58.6 ÷ 17.44 = 3.36 from Pt A

e = 4.5 − 3.36 = 1.14'

S.M = bh² ÷ 6 = 9² ÷ 6 = 13.5

$$\text{pressure} = \frac{V}{A} \pm \frac{Ve}{S.M} = \frac{17.44}{9} \pm \frac{17.44 \times 1.14}{13.5} =$$

$$= 1.94 \pm 1.47 = 3.26\ k/\square$$

0.47 k/□

ok less than 4 k/□

HNTB

MADE BY C D DATE 9/11/75 JOB NO. 9104
CHECKED BY JAJ DATE 4 22 75 SEC. NO.
SHEET NO. 5-56

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

STEM STEEL

Mum @ Base = 1.8 × 8 + .4 × 5 × 2.5 = 19.4 '^k/'

V @ Base = 1.8 + .4 × 5 = 3.8 ^k/'

Mum 5' up = 1.8 × 3 = 5.4 '^k/'

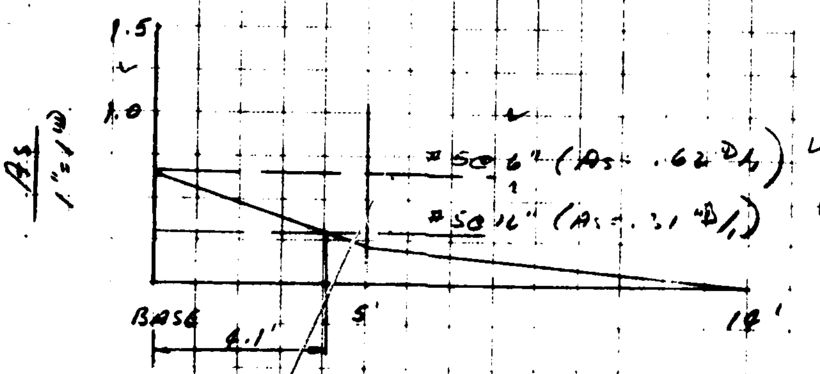
V 5' up = 1.8 ^k/'

d @ Base = 26 - 4 - .5 = 21.5"

d 5' up = (12 + 9) - 4 - .5 = 16.5"

As Base = 19.4 × 12 ÷ 20 × .891 × 21.5 = 0.61 □"/'

As 5' up = 5.4 × 12 ÷ 20 × .891 × 16.5 = 0.22 □"/'



Cut-off of Long Dowel = 4.1 + d = 4.1 + 16.5/12 = 5.5'
Use 12 since .8 × 10.0 < 12"
Splice Length #5 = 1.3 × 12" = 15.6" → Use 16"

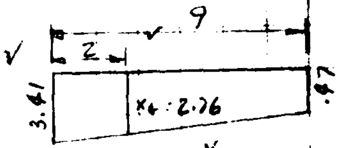
Ref ACI-318-71 Pg 48 #12.5

HNTB

MADE BY C.D. DATE 9/13/75 JOB NO. 4104
CHECKED BY J.K.T. DATE 9.22.75 SEC. NO.
SHEET NO. 5-57

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

Toe Steel (Surcharge)



$x_4 = .47 + \frac{7}{9}(3.41 - .47)$

$x_4 = 2.76$

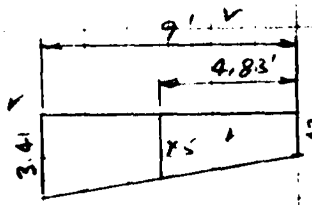
		V		M
	3.41 × 2 ÷ 2	= 3.41	× 1.33 =	4.5
	2.76 × 2 ÷ 2	= 1.76	× 0.67 =	1.8
Conc	2 × 2 × .15	= −0.60	× 1	−0.6
Earth	5 × 2 × .133	= −1.33	× 1	−1.3
		+4.24 k/l		+4.4 k/l

$d = 20 - 4 - \frac{1}{2} = 15.5''$

$A_s = 4.4 \times 12 \div 20 \times .891 \times 15.5 = .15$ □''/l

Use #5 @ 12" (Bend from Stem)
(A_s = .31 □''/l)

Heel Steel



		V		M
	2.05 × 4.83 ÷ 2 =	4.95	× 1.61 =	8.0
	0.47 × 4.83 ÷ 2 =	1.14	× 3.22 =	3.7
Conc	2 × 4.83 × .15 =	−1.45	× 2.42 =	−3.5
Earth	4.83 × 14 × .133 =	−8.99	× 2.42 =	−21.8
		−4.35 k/l		−13.6 k/l

$x_5 = .47 + \frac{4.83}{9}(3.41 - .47)$

$x_5 = 2.95$

$A_s = 13.6 \times 12 \div 20 \times .891 \times 15.5 = .47$ □''/l

Use #7 @ 12" (A_s = .60 □''/l)

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

HNTB

MADE BY C.D DATE 1/18/75 JOB NO. 3104
CHECKED BY JKJ DATE 4.23.75 SEC. NO.
SHEET NO. 5-58

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

TEMP & SHRINKAGE REINF

REF. E.M. 1110-2-2103 Pg 5 b(3)

MIN. RESTRAINED @ END CASE

$As/\text{face} = \frac{.004}{2}(1+2.12) \times \frac{12}{2} \times 12 = .46^{\square''}/'$

USE #7 @ 12" ($As = .60^{\square''}/'$) FOR LOWER 3' OF GATE

FOR REMAINDER HORIZ STEEL Pg 5 b(1)

$As/\text{face} = \frac{.0025}{2}(1+2.12) \times \frac{12}{2} \times 12 = .29^{\square''}/'$

USE #5 @ 12" ($As = .31^{\square''}/'$)

HNTB

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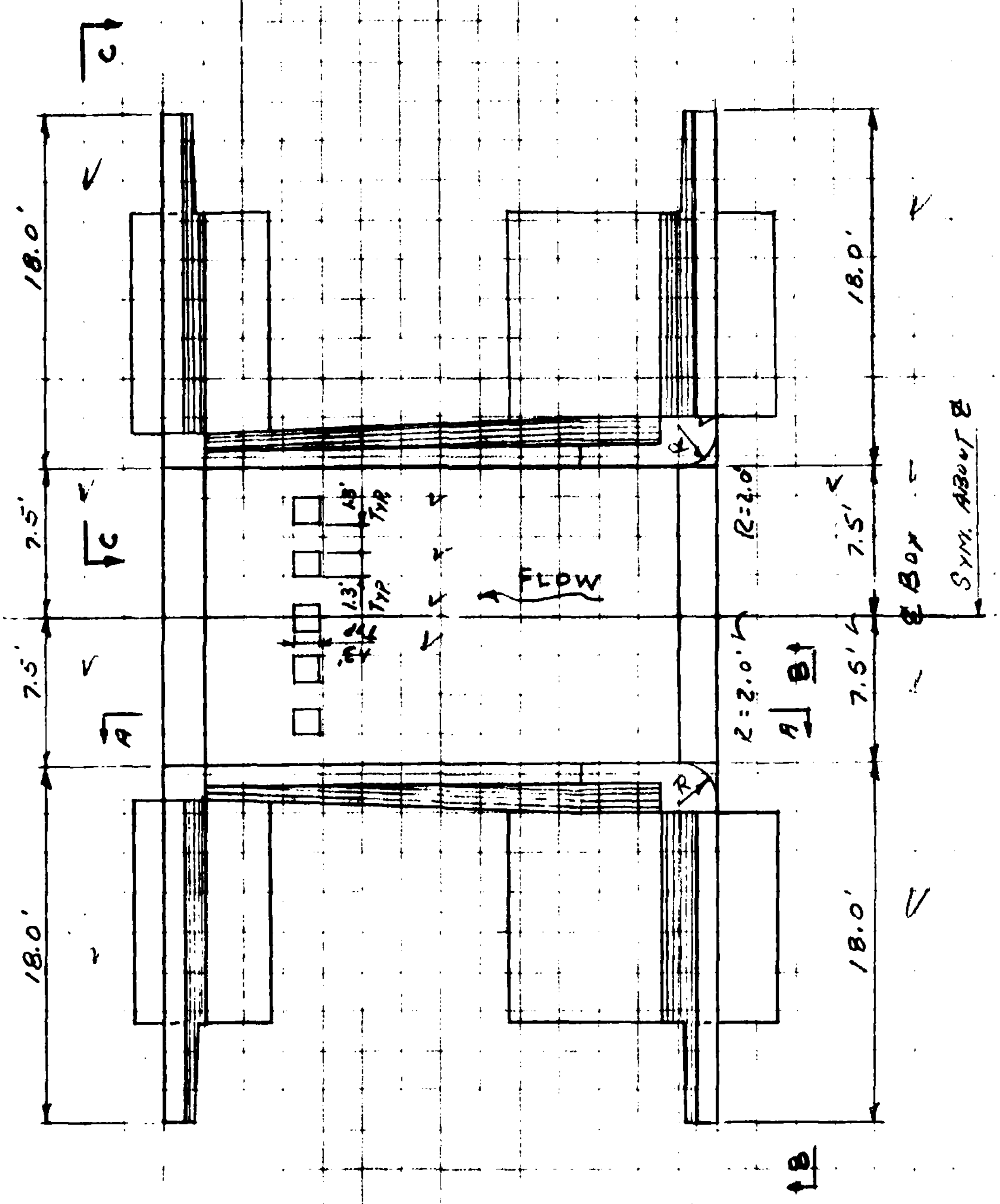
MADE BY C D DATE 9/11/75 JOB NO. 92-4
CHECKED BY J A J DATE 4-23-75 SEC. NO.
SHEET NO. S-58

CALCULATIONS FOR Coy Glen, Ithaca



SCHEME No. 1
(DRAW. STR. Nos 1 & 2)
1/8" = 1'-0"

NOTE:
(1) FOR CONSTRUCTION PROCEDURE SEE SH. 58A.

71

HNTB

MADE BY C.D. DATE 3/6/75 JOB NO. 4204
CHECKED BY JkJ DATE 4.23.75 SEC. NO.
SHEET NO. S-59A

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

CONSTRUCTION PROCEDURE SCHEME NO 1

1) EXCAVATE FOR BOX CONSTRUCTION ✓
(ELEV 383.3 DROP STR NO 1) ✓
(" 370.5 " " " 2) ✓

2) CONSTRUCT BOX ✓

3) EXCAVATE FOR WALL CONSTRUCTION ✓
(DROP STR NO 1 TO BE EXCAVATED TO
✓ ELEV 375, THEN BACKFILLED WITH SUITABLE
MATERIAL TO ELEV 383.3)
(DROP STR NO 2 TO BE EXCAVATED TO
ELEV. 370.5) ✓
✓ NOTE: CARE SHALL BE EXERCISED SO AS
NOT TO UNDERMINE THE NEWLY
CONSTRUCTED BOX

4) CONSTRUCT WALLS. ✓

HNTB

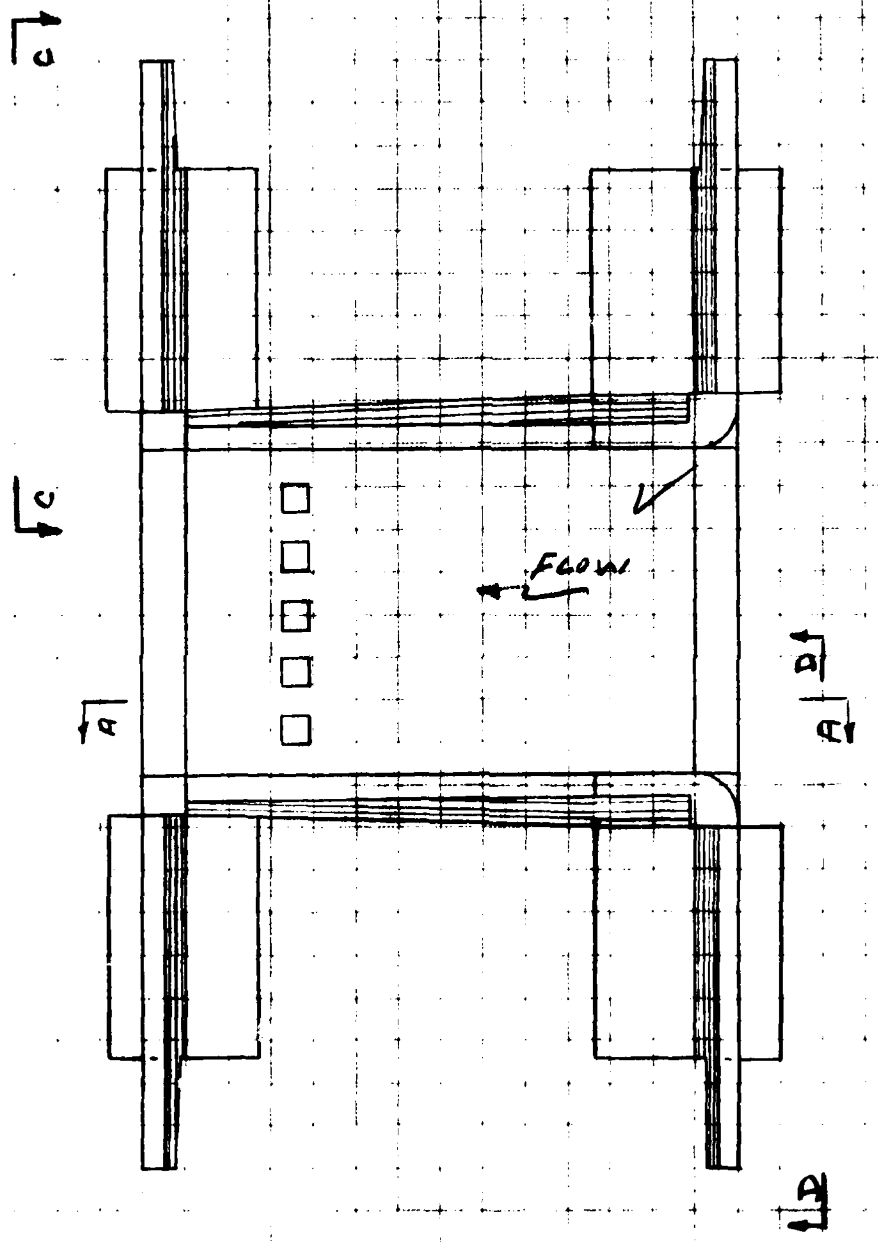
MADE BY C.D DATE 9/18/75 JOB NO. 4114

CHECKED BY J K J DATE 4 27 75 SEC. NO.

SHEET NO. 5-59B

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



SCHEME No 2
(DROP STR No 1 & 2)
1/8" = 1'-0"

NOTE:

(1) FOR CONSTRUCTION PROCEDURE SEE SH 59C

(2) FOR DIM. SEE SH 59.

HNTB

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

MADE BY C.D. DATE 2/6/75 JOB NO. 4204
CHECKED BY J.K.J DATE 4.23.75 SEC. NO.
SHEET NO. 5-59C

CONSTRUCTION PROCEDURE SCHEME NO 2

1) EXCAVATE FOR BOX CONSTRUCTION
(ELEV 383.3 DROP STR NO 1)
(" 379.5 " " " 2)

2) CONSTRUCT BOX

3) EXCAVATE FOR WALL CONSTRUCTION.
(DROP STR NO 1 TO BE EXCAVATED TO ELEV 375, THEN BACKFILL UPSTREAM WALLS TO ELEV 388.3, AND DOWNSTREAM WALLS TO ELEV. 383.3 WITH SUITABLE MATERIAL)
(DROP STR NO 2 TO BE EXCAVATED TO ELEV 371.5, THEN BACKFILL UPSTREAM WALLS TO ELEV 379.5 WITH SUITABLE MATERIAL)
NOTE: CARE SHALL BE EXERCISED SO AS NOT TO UNDERMINE THE NEWLY CONSTRUCTED BOX.

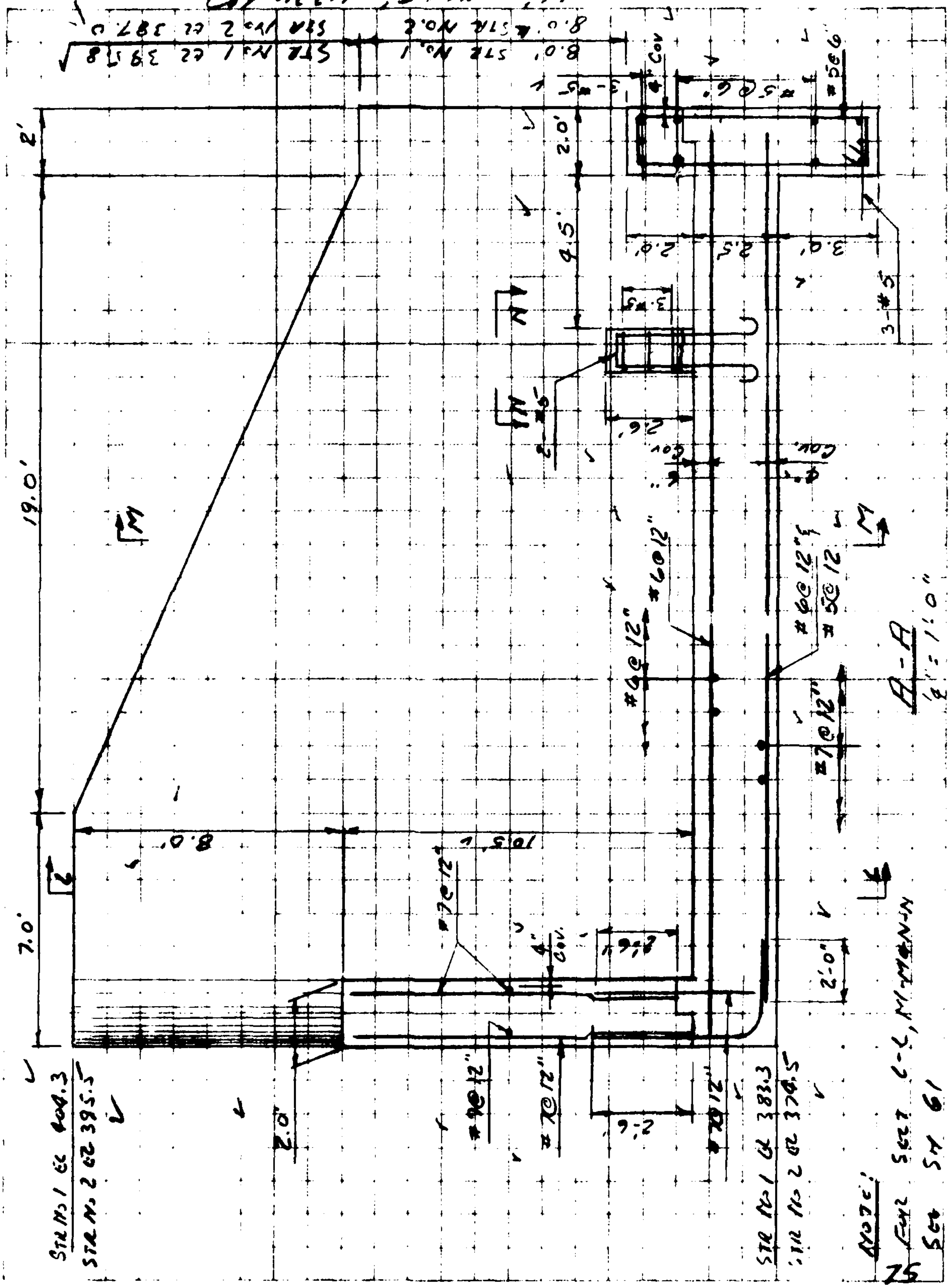
4) CONSTRUCT WALLS.

HNTB

MADE BY ___ DATE ___ JOB NO. ___
CHECKED BY JKJ DATE 4 23 75 SEC. NO. ___
SHEET NO. 5-60

CALCULATIONS FOR ___

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



HNTB

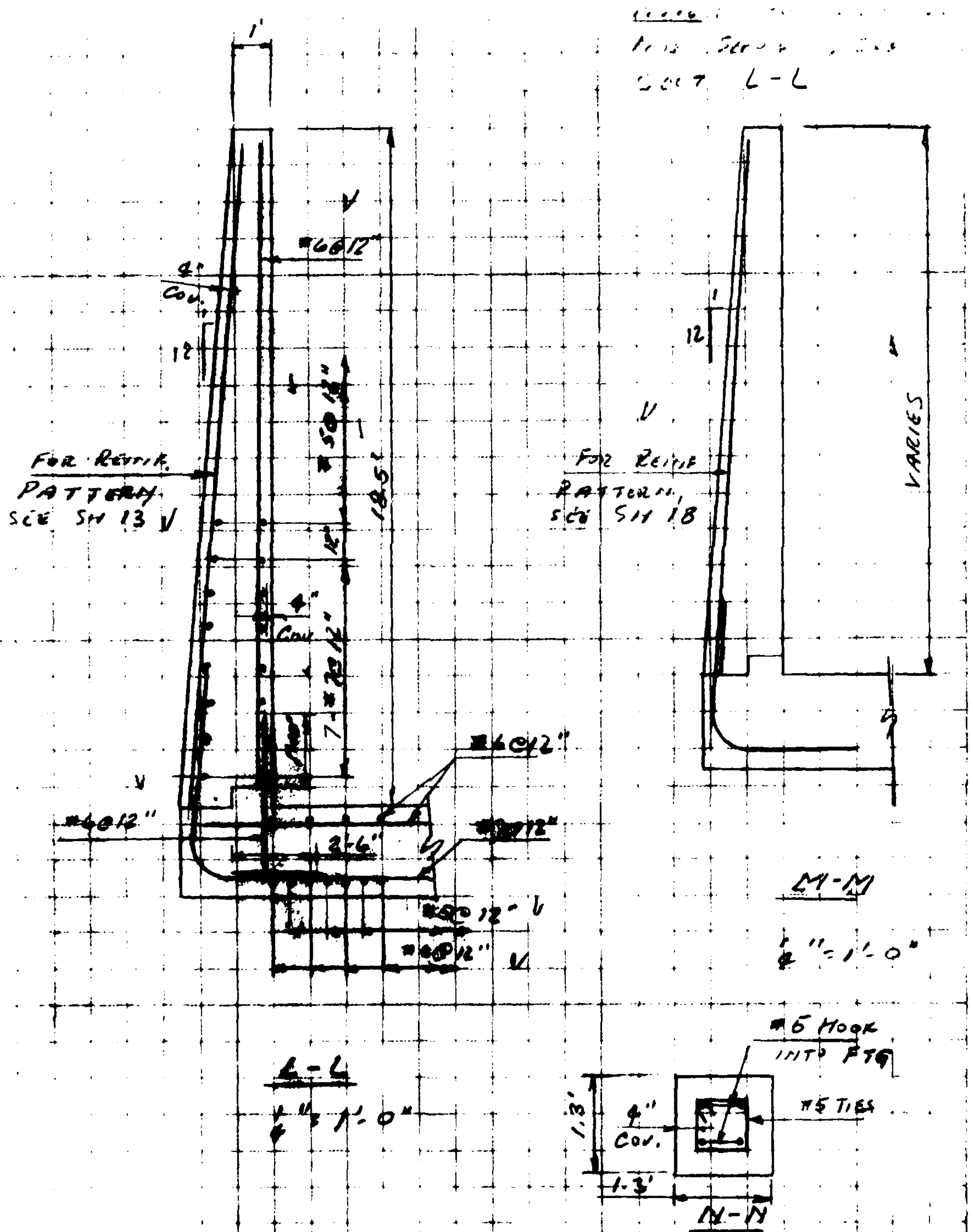
HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

MADE BY L.L. DATE 4/16/75 JOB NO. 4204

CHECKED BY J.K.J. DATE 4.23.75 SEC. NO.

SHEET NO. 5-61

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

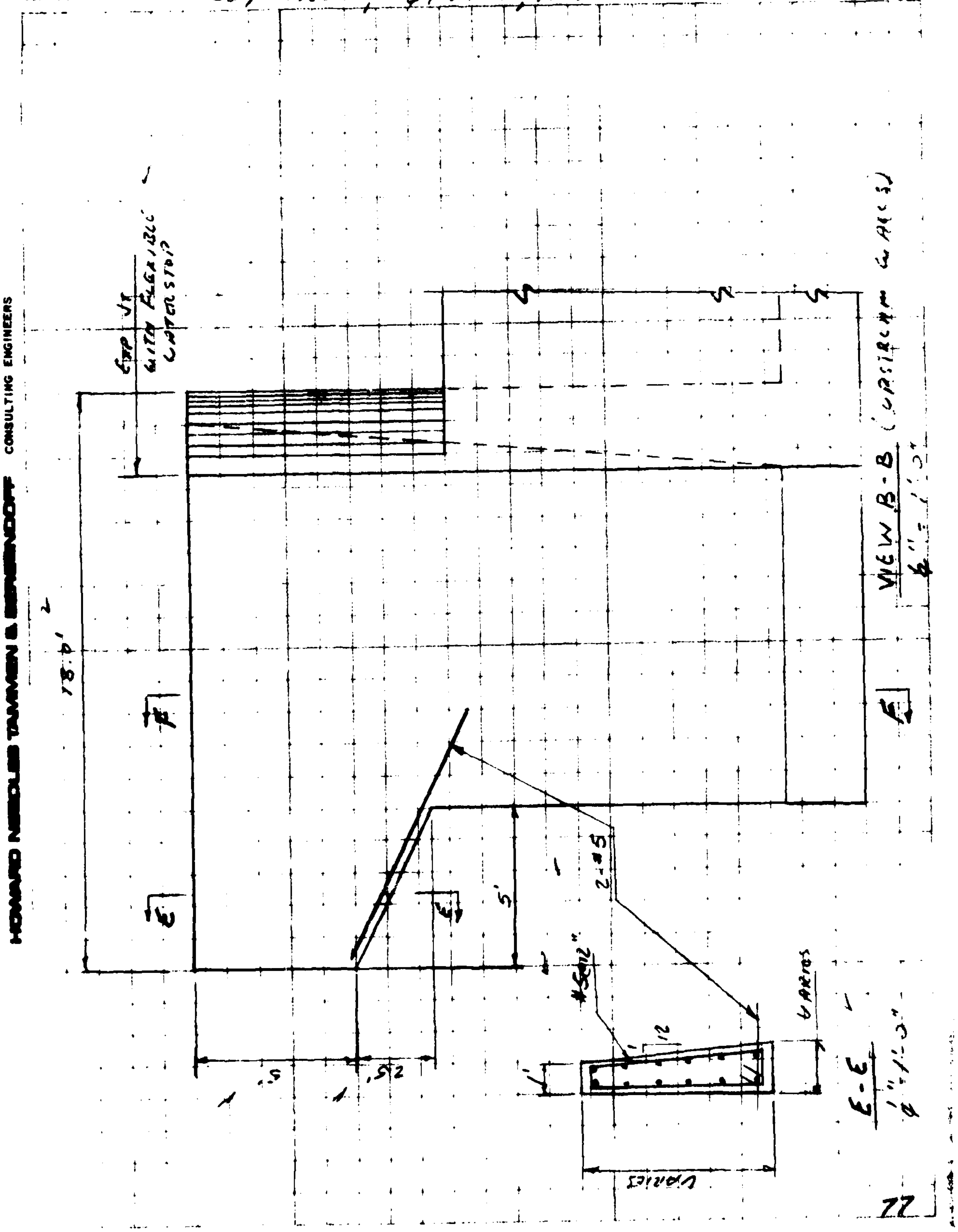


HNTB

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

MADE BY C.D DATE 7/16/77 JOB NO. 4104
CHECKED BY JKJ DATE 4.23.75 SEC. NO.
SHEET NO. 5-62

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.



HNTB

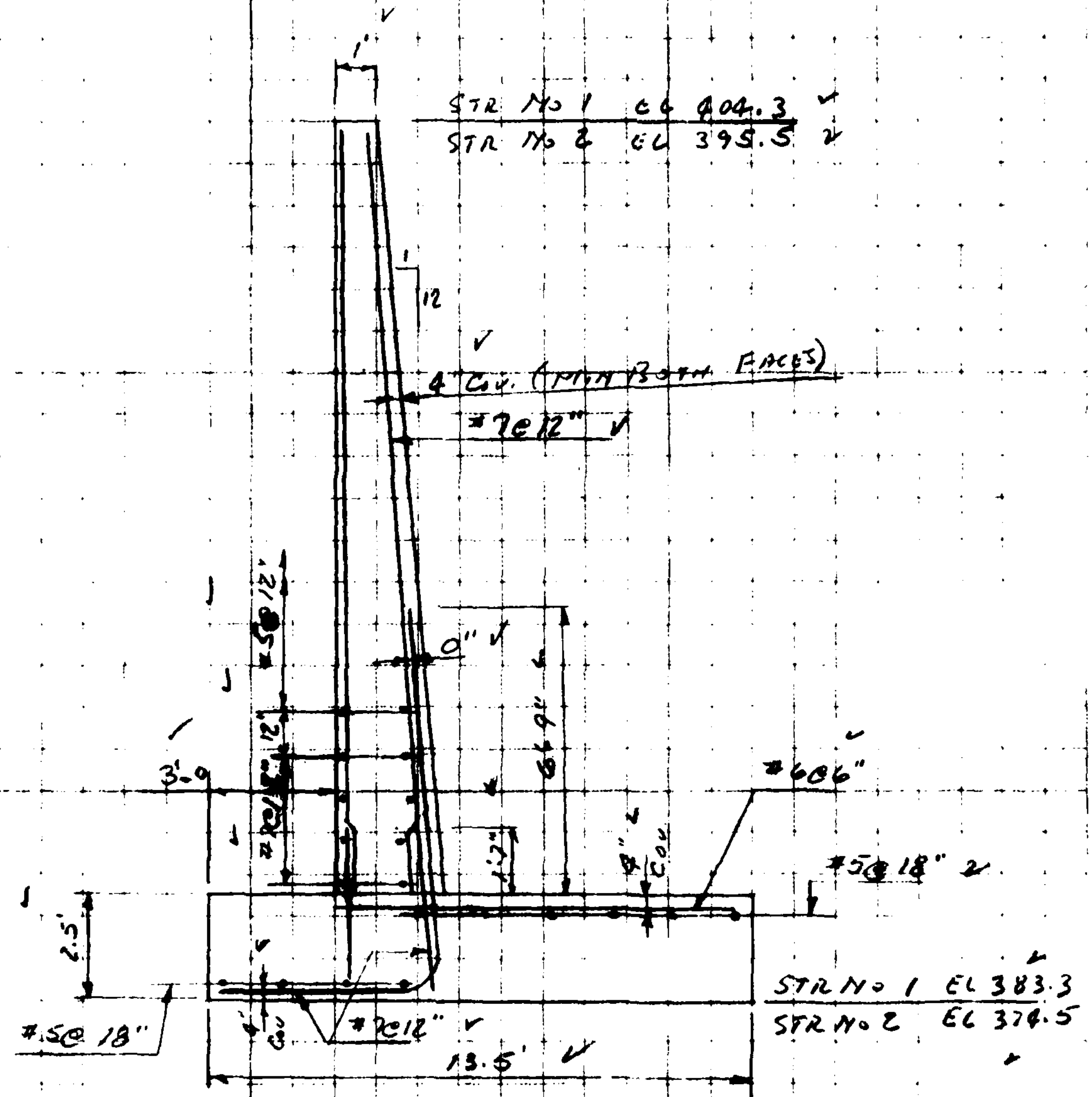
MADE BY ___ DATE ___ JOB NO. 9104

CHECKED BY JAZ DATE ___ SEC. NO. ___

SHEET NO. 5-63

CALCULATIONS FOR Cay Geen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS



E-F

1" = 1'-0"

HNTB

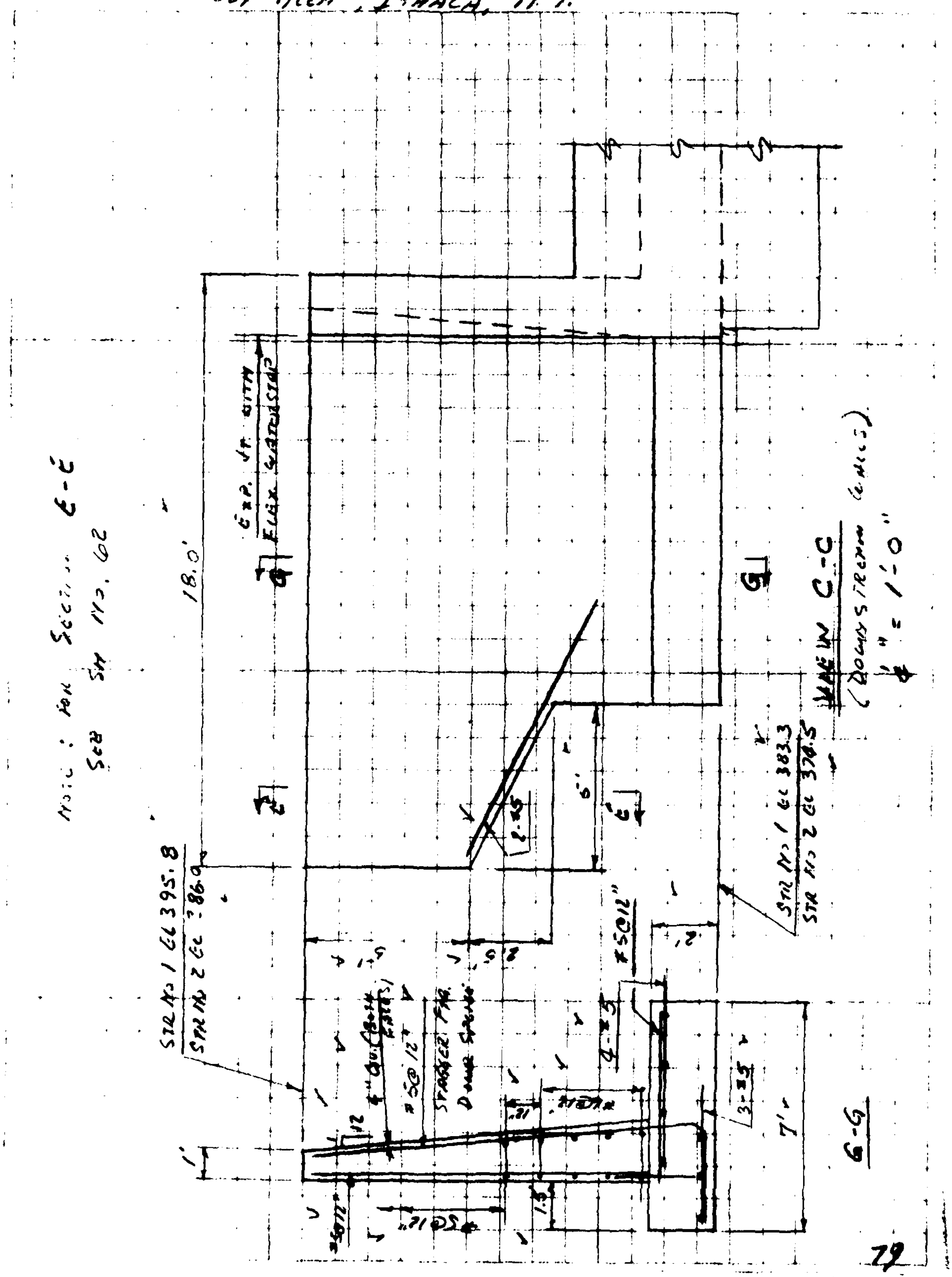
CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

MADE BY ___ DATE 9/17/75 JOB NO. 9104

CHECKED BY J.A.T. DATE 4.24.75 SEC. NO. ___

SHEET NO. 5-64

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

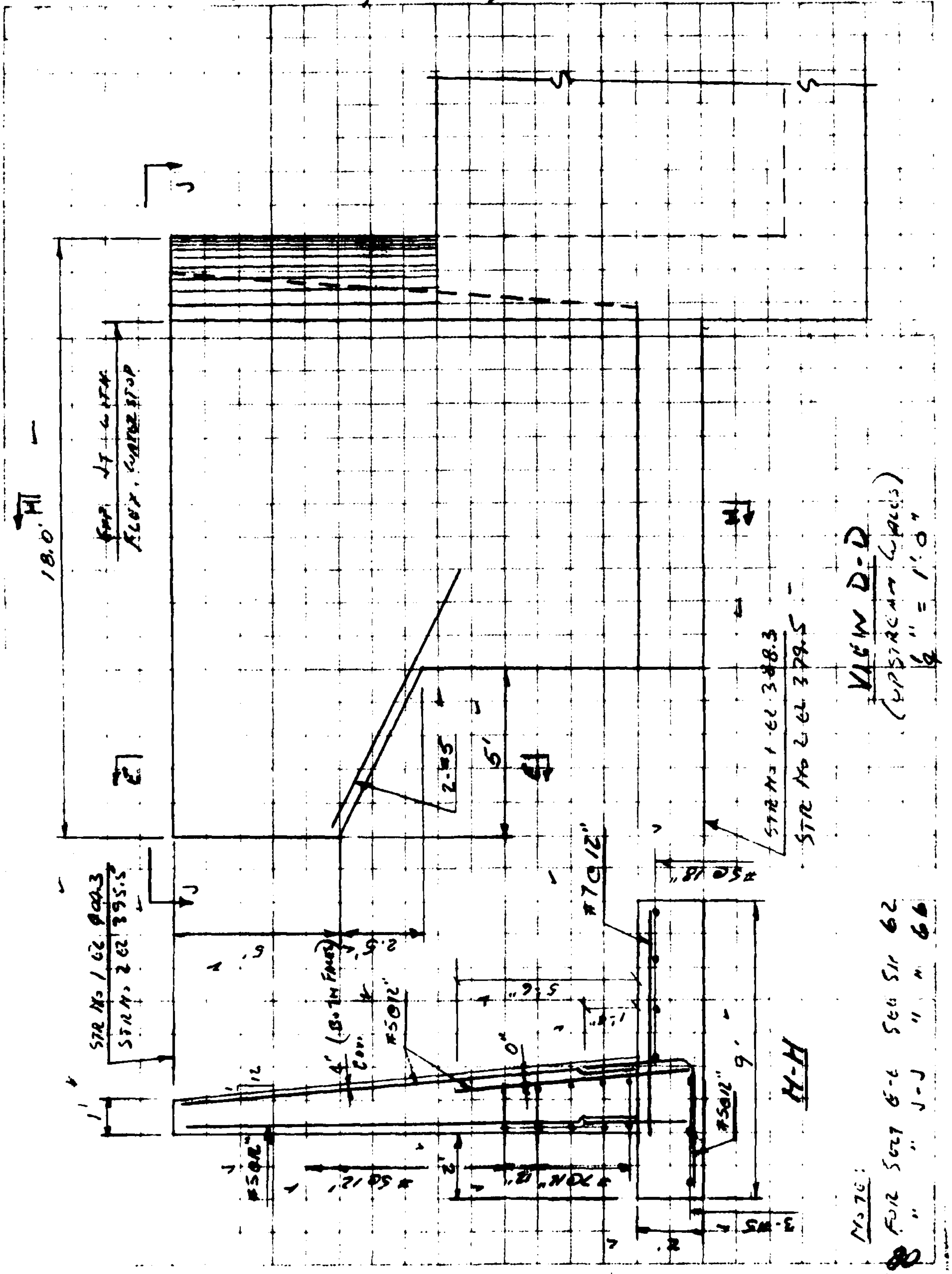


HNTB

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

MADE BY L.D. DATE 8/12/75 JOB NO. 8204
CHECKED BY ... DATE 4.29.71 SEC. NO.
SHEET NO. 5-65

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.



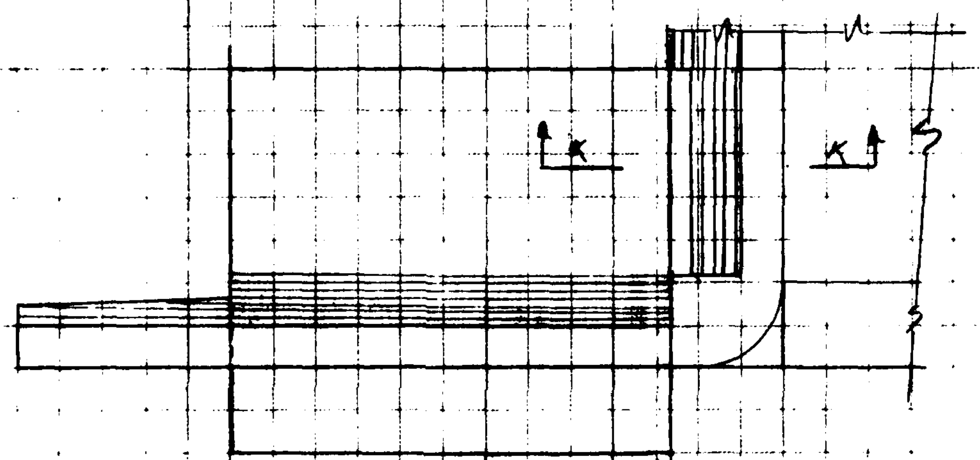
HNTB

MADE BY C.D DATE 4/18/75 JOB NO. 4604

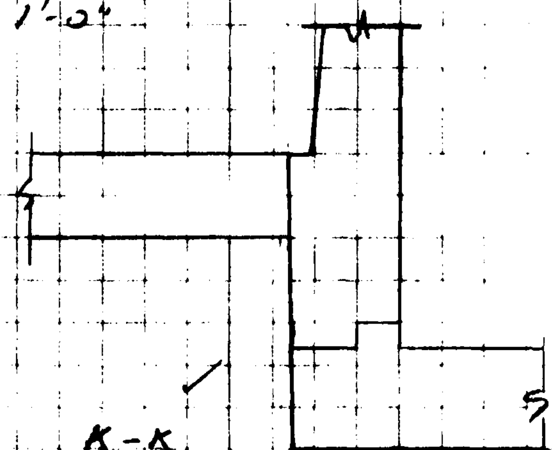
CHECKED BY JKJ DATE 4.24.75 SEC. NO.

SHEET NO. 5-66

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.



VIEW J-J
(UPSTREAM WALL)
$\frac{1}{4}'' = 1'-0''$



K-K
$\frac{1}{4}'' = 1'-0''$

2. SOILS ANALYSIS

2.1 This section outlines the methods used to determine the lateral pressures for design of the drop structures, the stability of the drop structure excavation, and the design of the sheet pile alternate for the drop structure wingwalls.

2.2 For the design of the drop structures, at-rest earth pressures were used while for the design of the wingwalls, active pressures were used, Sheet A-1. The backfill for which these pressures were calculated was a sand having a natural unit weight of 120 pcf, a saturated unit weight of 133 pcf and an angle of internal friction of 30 degrees.

2.3 The soil profile for these designs is based on borings made for the Cayuga Inlet Flood Protection construction and is shown on Sheet A-2.

2.4 The shear strength of the clay above El. 375 is important to the design of the excavations for the two drop structures and the sheet pile wingwall alternates. From spoon penetration resistances, average N-2.5, it was initially estimated that the cohesion of the clay was 312 psf, Sheet A-5. Subsequently a limited number of torevane and pocket penetrometer tests were made on the clay at the site, Sheet A-7, from which it was concluded that the value of the clay cohesion was 600 psf or greater.

2.5 The stability of the excavations for the cohesion values of 312 psf and 600 psf were checked, Sheets A-5 and A-5a. For a cohesion value of 600 psf and a safety factor of 1.5, a 0.5H:1V slope can be used

for Structure No. 1, and a 1H:1V slope can be used for Structure No. 2. However, because of potential seepage problems at the excavation bottom for Structure No. 2 and potential bottom heave at Structure No. 1, it is recommended that dewatering of the sand and gravel layer below El. 375 be used at both structures.

2.6 Structure No. 2 will be founded on the dense sand and gravel stratum below El. 375 and therefore should have good bearing. The same condition applied for the concrete cantilever wingwall alternates for this structure which will be founded at the same level. Structure No. 1 will be founded on clay at El. 383. The normal dead load bearing capacity safety factor for the box structure is 4.0 and the minimum safety factor for dead plus maximum live load is 2.9. For the concrete cantilever wingwall alternate, the minimum safety factor ranges from 1.6 for the downstream wall to 1.0 for the upstream wall. Should it be desired to utilize the concrete cantilever wingwall alternate of this structure, it is recommended that the clay beneath the wall be removed down to the sand and gravel stratum at El. 375 and backfilled with sand or gravel to the footing elevation. Since the backfill will not be compacted, a maximum wingwall footing bearing value of 2 tsf should be used so to limit settlement of the wingwall. Bearing capacity calculations for a cohesion of 600 psf are given on Sheet A-7.

2.7 Cantilever steel sheet pile wall alternates were designed for both drop structures. Active earth pressures were utilized for these designs.

Above the channel bottom a granular backfill was assumed with full water pressure for the design channel depth of five feet. Below the channel bottom clay soil was assumed and the differential water level was assumed to decrease linearly to zero at El. 375 since the sand and gravel layer below this level is expected to balance the water pressures in each side of the sheeting. The design for the cantilever steel sheet pile wingwall alternates for both structures is included in Sheets A-8 to 31. Design summaries are given on Sheet A-32.

HNTB

MADE BY MS-J DATE 4-2-75 JOB NO. 4204-99-01
CHECKED BY CW DATE 5-6-75 SEC. NO.
SHEET NO. A-1

CALCULATIONS FOR Gay Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Soil Pressures for Structure Design.

Str. No	El. @ top	El. @ bottom (Ref. Inc. D-2)
1	401	384 ±
2	393	375 ±

Soil Conditions (Ref borings SS-D & SS-1) (See Sh. A-2)

El. 401 to 384 CL, N = 1-4 2.5 av

Est. NWC = 30% (Ref. Boring SS-D)

For NWC = 30%, $G_s = 2.67$ $\gamma_{dry} = 93$ pcf (Sh. A-3)

$\gamma_{sat} = 1.30 \times 93 = 121$ pcf.

$\gamma_{buoyant} = 121 - 62.4 = 58.6$ pcf

El. 393 to 375

El. 393 to ~~383~~ 375 CL as above

below 375 ~~383 to 375~~ Sand, Gravel GP GM N = 11 to 38, av = 23

$D_r = 40\%^*$, $\gamma_0 = 100$ pcf*, $\gamma_{100} = 140$ pcf* $\gamma_{40} = 113$ pcf, (Sh. A-3)

Sat. NWC = 18% (Sh. A-3), $\gamma_{sat} = 113 \times 1.18 = 133$ pcf

$\gamma_{buoy} = 133 - 62.4 = 70.6$ pcf

* See Sh. A-4

For Wall Design (U Wall) use K_0 (EM 1110-2-2502 4.c(4), p5)

For $\phi = 30°$, $K_0 = 1 - \sin\phi$, $K_0 = 1 - 0.5 = 0.5$

Design lateral Earth Pressures for Box.

1) Saturated Unit Wt.

$p_s = 133 \times 0.5 = 66.5$ psf/ft.

2) Boyant Unit Wt.

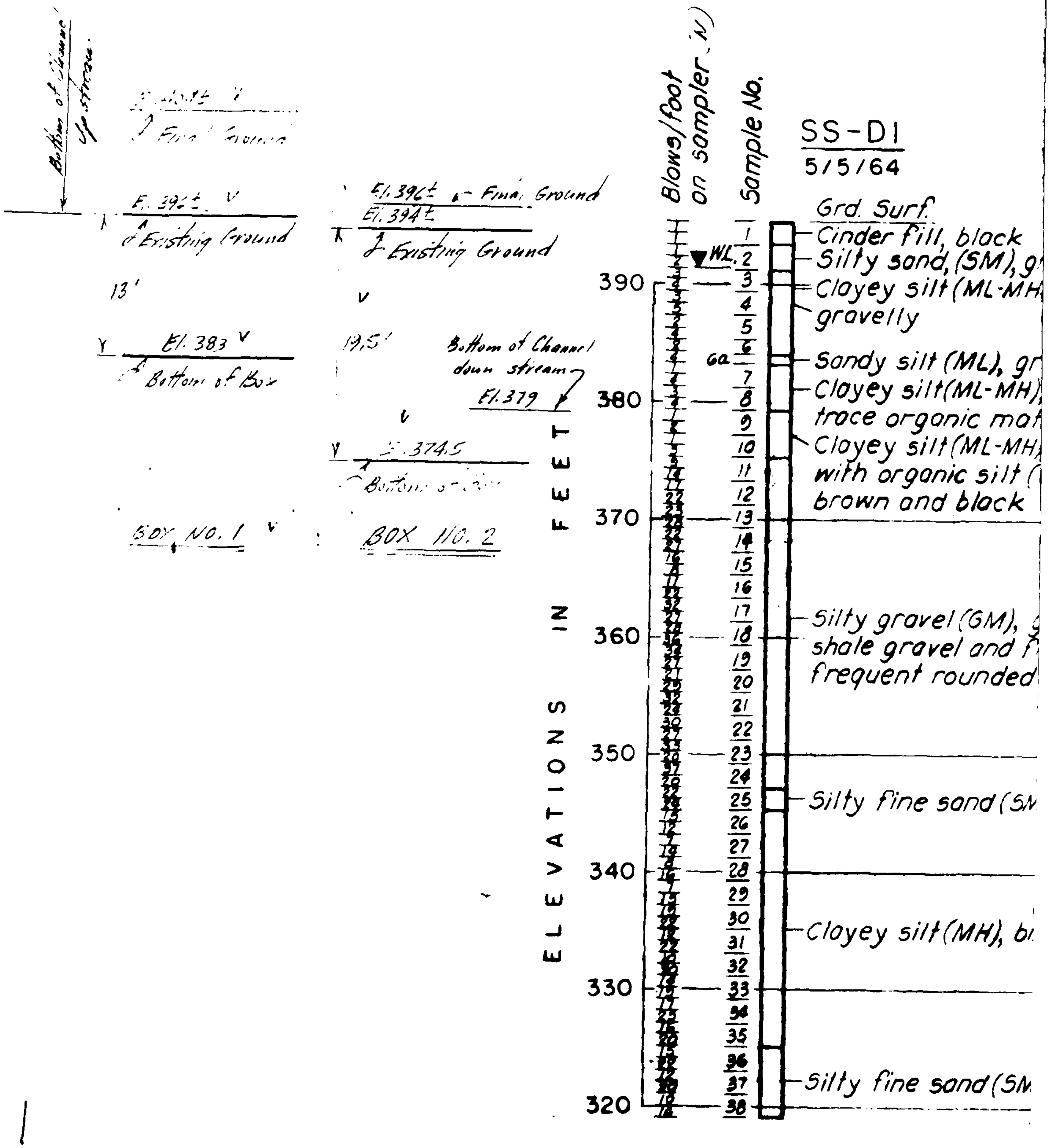
$p_b = 70.6 \times 0.5 = 35.3$ pcf/ft.

Active Lateral Earth Pressures for Wingwalls for $\phi = 30°$ $K_a = 0.333$

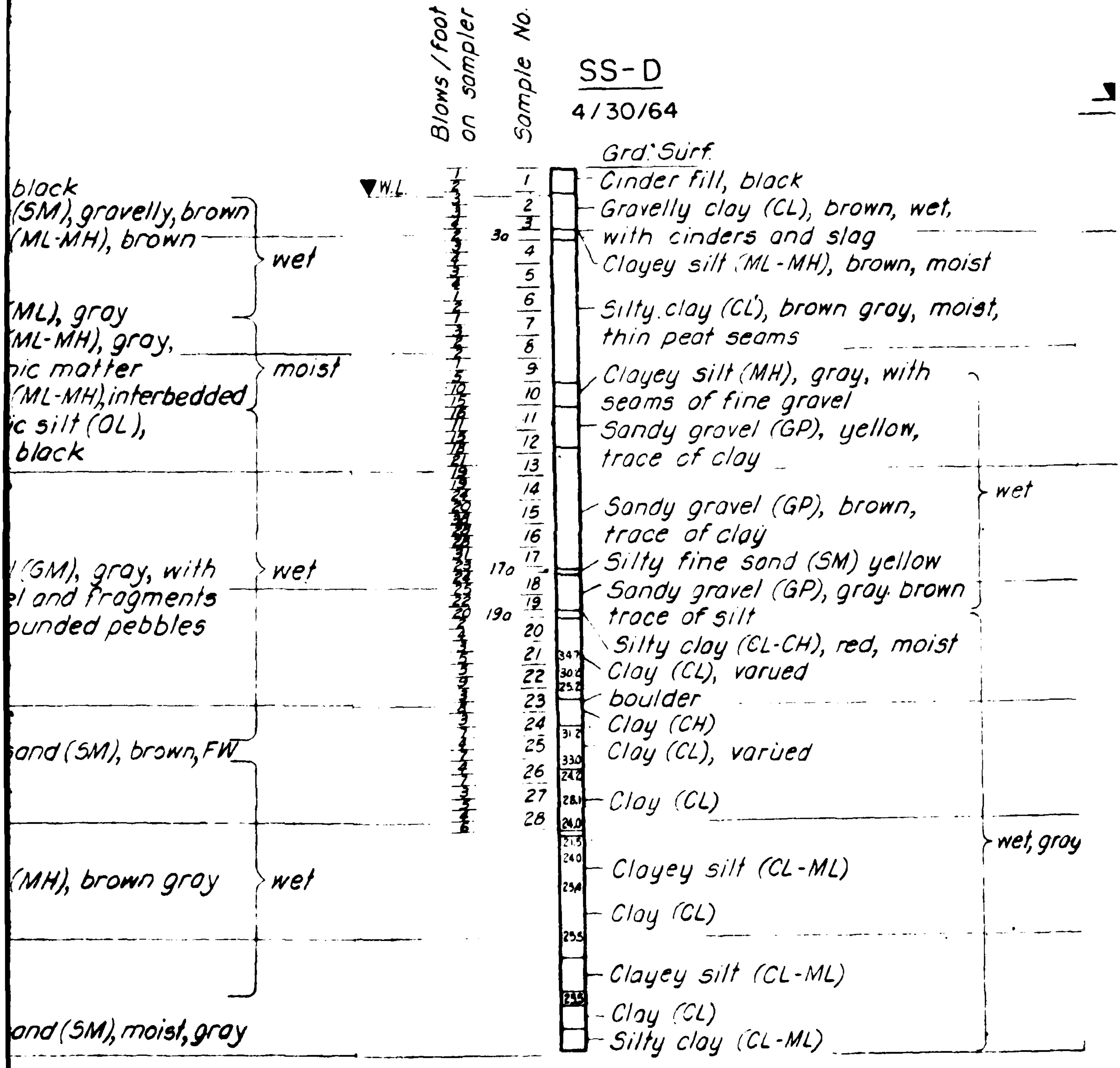
1) Saturated $p_s = 133 \times 0.333 = 44.3$ psf

2) Boyant $70.6 \times 0.333 = 23.5$ psf

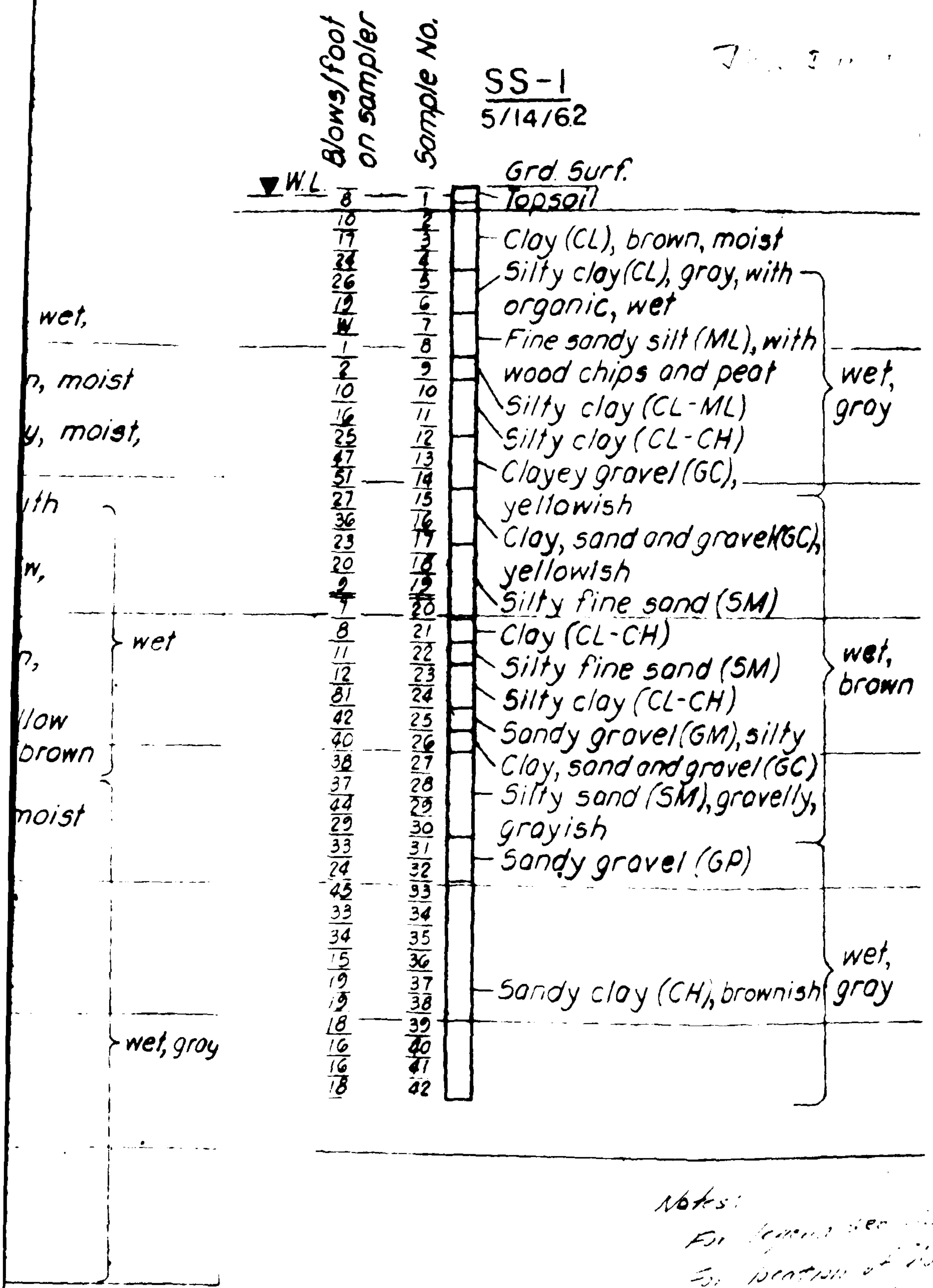
85



Bottom of Channel Up stream
El. 394± Existing Ground
13'
El. 383 Bottom of Box
BOX NO. 1
El. 396± Final Ground
El. 394± Existing Ground
19.5'
Bottom of Channel down stream
El. 379
374.5
BOX NO. 2
ELEVATIONS IN FEET
390
380
370
360
350
340
330
320
Blows/foot on sampler (N)
Sample No.
WL
6a
SS-D1
5/5/64
Grd. Surf.
Cinder fill, black
Silty sand, (SM),
Clayey silt (ML-MH
gravelly
Sandy silt (ML), gr
Clayey silt(ML-MH),
trace organic mat
Clayey silt(ML-MH)
with organic silt
brown and black
Silty gravel (GM),
shale gravel and
frequent rounded
Silty fine sand (SM
Clayey silt (MH), b
Silty fine sand (SM



black
(SM), gravelly, brown
(ML-MH), brown
wet
(ML), gray
(ML-MH), gray,
nic matter
moist
(ML-MH), interbedded
ic silt (OL),
black
(GM), gray, with
el and fragments
ounded pebbles
wet
and (SM), brown, FW
(MH), brown gray
wet
and (SM), moist, gray
W.L.
Blows / foot on sampler
Sample No.
SS-D
4/30/64
Grd. Surf.
Cinder fill, black
Gravelly clay (CL), brown, wet, with cinders and slag
Clayey silt (ML-MH), brown, moist
Silty clay (CL), brown gray, moist, thin peat seams
Clayey silt (MH), gray, with seams of fine gravel
Sandy gravel (GP), yellow, trace of clay
Sandy gravel (GP), brown, trace of clay
Silty fine sand (SM) yellow
Sandy gravel (GP), gray brown trace of silt
wet
Silty clay (CL-CH), red, moist
Clay (CL), varved
boulder
Clay (CH)
Clay (CL), varved
Clay (CL)
Clayey silt (CL-ML)
Clay (CL)
Clayey silt (CL-ML)
Clay (CL)
Silty clay (CL-ML)
wet, gray
1
2
3
3a
4
5
6
7
8
9
10
11
12
13
14
15
16
17
17a
18
19
19a
20
21
22
23
24
25
26
27
28
34.7
30.6
25.2
31.2
33.0
24.2
28.1
24.0
21.5
24.0
23.4
25.5
23.5



Notes:
For legend see Sh. A-2a
For location of borings
see Sh. A-2b

LEGEND -(SUBSURFACE EXPLORATIONS)

5/27/64	- DATE EXPLORATION WAS COMPLETED
▼ W.L.	- WATER LEVEL IN HOLE WHEN EXPLORATION WAS MADE
150(0.4')	- BLOWS PER FRACTION OF FOOT AS INDICATED
$\boxed{30\%}$	- MOISTURE CONTENT, % DRY WEIGHT
FW	- FREE WATER IN JAR SAMPLE
SP	- SAND, POORLY GRADED
SM	- SAND, SILTY
SC	- SAND, CLAYEY
GC	- GRAVEL, CLAYEY
GM	- GRAVEL, SILTY
GP	- GRAVEL, POORLY GRADED
GW	- GRAVEL, WELL GRADED
ML	- INORGANIC SILT, LOW TO NO PLASTICITY
MH	- INORGANIC SILT, HIGH PLASTICITY
ML-MH	- SILT, BORDERLINE OR MEDIUM PLASTICITY
CL-ML	- BORDERLINE BETWEEN CLAY AND SILT
SM-ML	- BORDERLINE BETWEEN SILTY SAND AND SANDY SILT
CL	- INORGANIC CLAY, LEAN, LOW TO MEDIUM PLASTICITY
CH	- INORGANIC CLAY, FAT, HIGH PLASTICITY
CL-CH	- BORDERLINE BETWEEN LEAN AND FAT CLAY
OL	- ORGANIC SILT OR CLAY, LOW TO MEDIUM PLASTICITY
PT	- PEAT, PREDOMINANTLY ORGANIC
N	- NO BLOW COUNT OR SAMPLE TAKEN
P	- SAMPLER PUSHED BY HAND
W	- SAMPLER SANK UNDER WEIGHT OF RODS AND HAMMER ALONE
NR	- NO RECOVERY
— — →	- INDICATES THE APPROXIMATE CHANNEL GRADE.

AD-A101 711 HOWARD NEEDLES TAMMEN AND BERGENDOFF NEW YORK F/G 13/13
ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR, COY GLEN AND CA--ETC(U)
AUG 75 DACW49-75-C-0052

UNCLASSIFIED NL

2 OF 3

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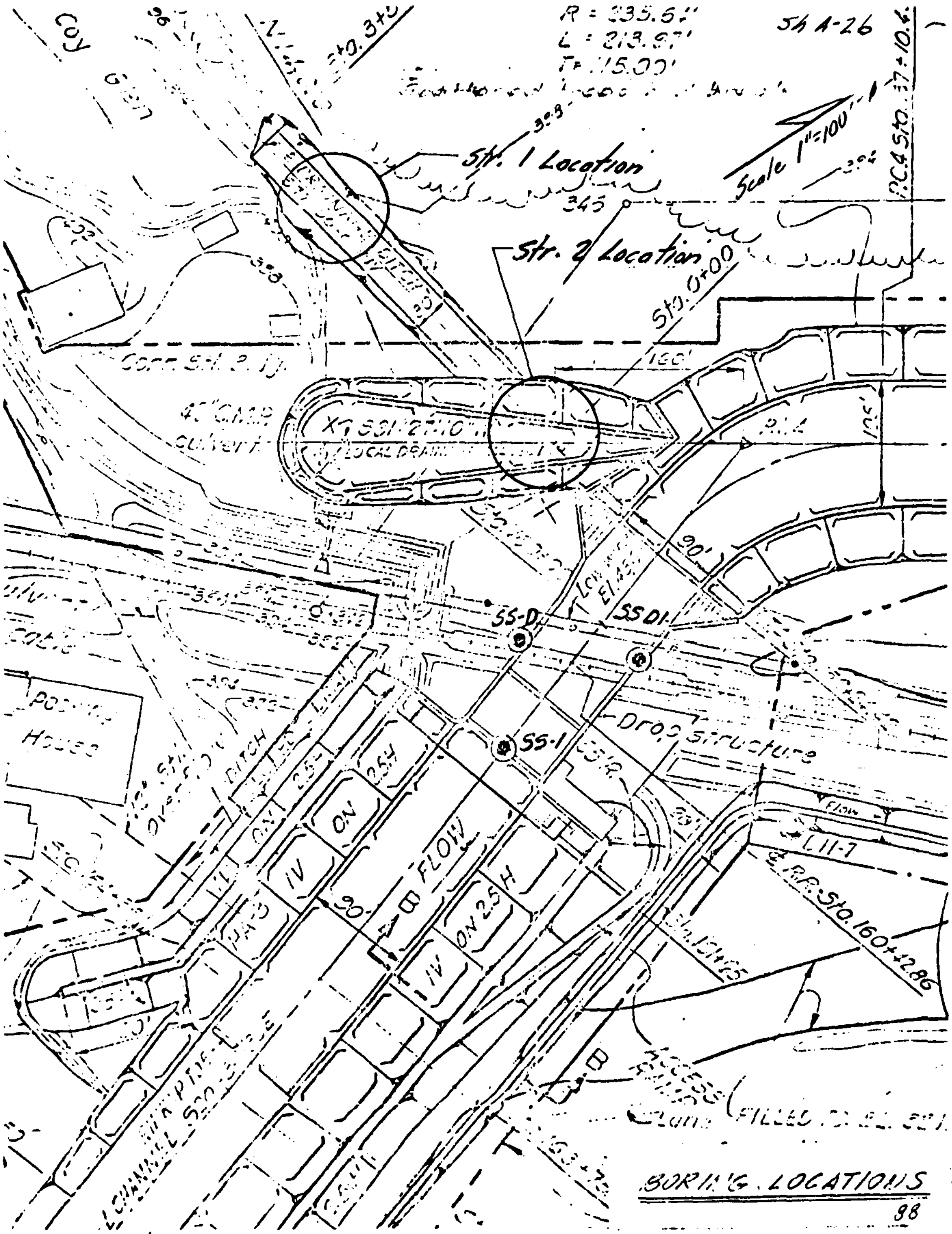
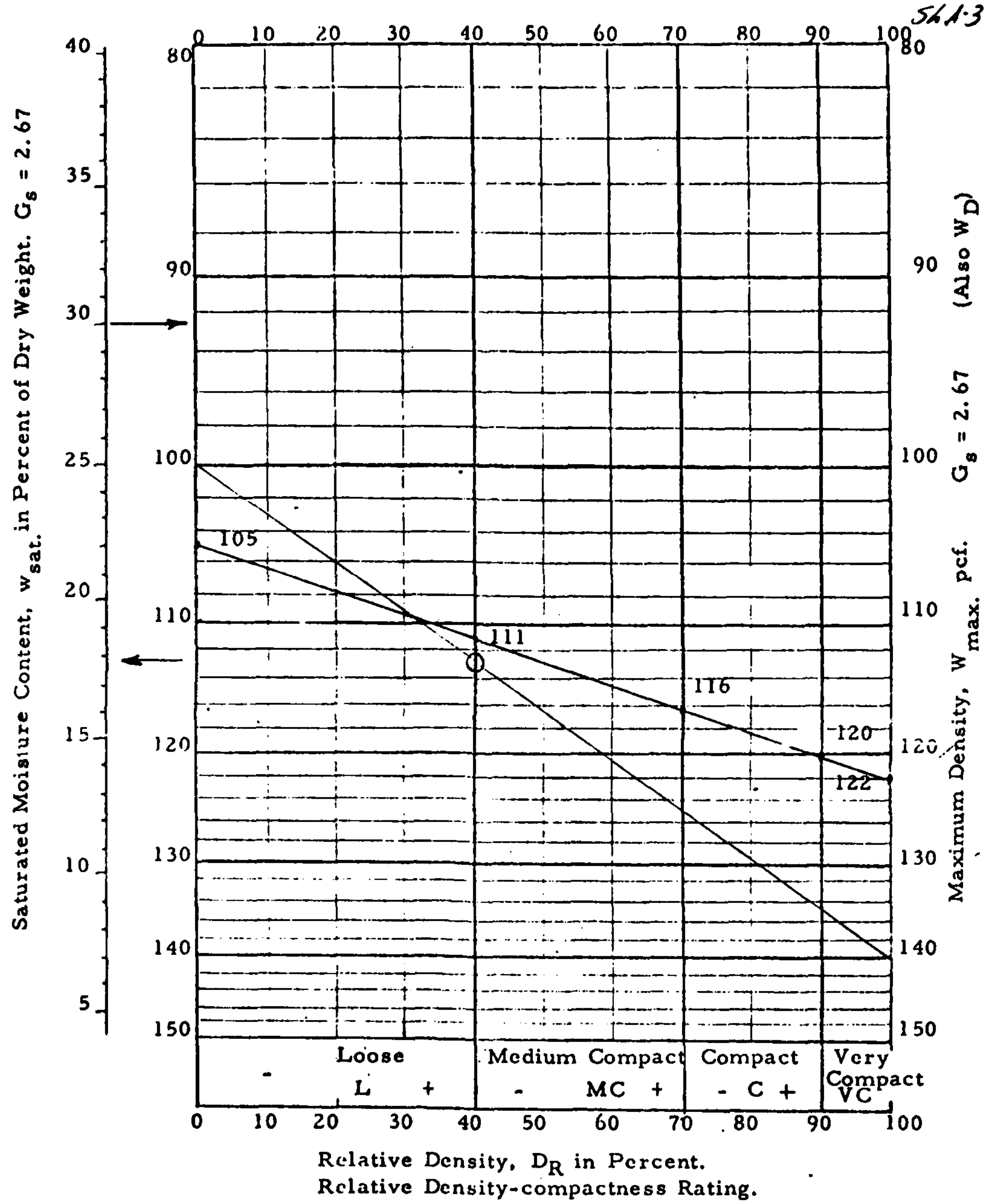


BORING LOCATIONS



$$D_R = \frac{e_L - e_N}{e_L - e_D} 100\% = \frac{1/W_L - 1/W_N}{1/W_L - 1/W_D} 100\% = \frac{w_L - w_N}{w_L - w_D} 100\%$$

FIG. 2 - Relative Density, Unit Weight, and Compactness Rating Diagram.

HNTB

MADE BY AAS-J DATE 4-2-75 JOB NO. 4204
CHECKED BY LOU DATE 5-6-75 SEC. NO.
CALCULATIONS FOR Coy Glen, Ithaca, N.Y. SHEET NO. A-4

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Unit Wt. of GP, GM soil (Sh. A-1)

$\gamma_{max} = \gamma_{100} = 135-145$ pcf (Table B1, Appendix B, "the Unified Soil Classification System, WES TM 3-357, 1957)

Use $\gamma_{100} = 140$ pcf

$\gamma_{min.} = \gamma_0 = 92$ to 115 pcf (Nos. 4-11, Table II "Physical, Stress-Strain and Strength Responses of Granular Soils" by D.M. Burmister, ASTM STP No. 322, 1962

Use $\gamma_0 = 100$ pcf

For $N = 23$, $D_r = 40\%$ (Fig. 6 D.M. Burmister reference)
also Sh. A-9

HNTB MADE BY AAS-T DATE 4-25-75 JOB NO. 4024
CHECKED BY [illegible] DATE 5-6-75 SEC. NO.
SHEET NO. A-5

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

Estimates of Shear Strength of Clay Soils [illegible]

	Boring SS-D1		Boring SS-D	
Elev 395 to 383	$\Sigma N = 33$, av $N = 33 \div 13 = 2.5$		$\Sigma N = 33$, a. $N = 33 \div 13 = 2.5$	
395 to 377±	48	$48 \div 19 = 2.5$	46	$46 \div 18 = 2.5$

From Terzaghi & Peck Table 45.2
for $N = 2$ $q_u = 0.25 tsf$ or $c = 250 psf$ or $c = 125 psf \cdot N$
Est. $c = 125 \times 2.5 = 312 psf$

note: varved clays from Hackensack Meadows
min $C = 500 psf$ for $N =$ push

Revised See Sh. A-7

Critical Depth of Excavation
For S.F. = 1.5 $C_d = 312 \div 1.5 = 208 psf$
For saturated clay use $\gamma_s = 121 pcf$ (Sh. A-1)
Check for 1H:1W and 1:1 excavation slopes, see sh A6

$i =$	63.5°	45°
$S_N =$	0.197	0.170
$H_c =$	8.7'	10.1'

$$H_c = \frac{C_d}{\gamma S_N} = \frac{208}{121 S_N} = \frac{1.72}{S_N}$$

Revised. see Sh. A-5A

Note: Depth of Excavation Box No. 1 = 16'
No 2 = 20'

Check for 2H:1W, $i = 26.5°$; $S_N = 0.153$, $H_c = 11.3'$

Check Bearing Capacity of Clay Soil (above Elev. 375)
$q = 5.14c = 5.14 \times 312 = 1600 psf$ (no surcharge)
surcharge effect, $D_f^* =$ El. 396 − 383 = 13'
use $\gamma' = \gamma_s = 121 - 62.4 = 58 pcf$
$D_f \gamma = 13 \times 58 = 755 psf$
Total $q' = q + D_f \gamma = 1600 + 755 = 2355 psf$
* Ref. Terzaghi & Peck 2nd Ed., page 222
Box: max load (Sh. 37) 2.0 ksf back, 0.66 ksf front
normal D.L. 0.50 ksf
Walls: toe (Sh. 44) 3.87 ksf

Revised. see Sh. A-7

Revised Critical Depth of Excavation:

For new value of $c = 600$ psf (See Sh. A-7)

For SF = 1.5, $c = 600$ psf, $C_d = \frac{600}{1.5} = 400$ psf

Slope	½H:1V	1H:1V	2H:1V
i	63.5°	45°	26.5°
S_N	0.197	0.170	0.153
H_c	16.8'	19.5'	21.7'

$$H_c = \frac{C_d}{\gamma S_N} = \frac{400}{121 S_N} = \frac{3.32}{S_N}$$

Note: Depth of Excavation: Box No. 1 = 16'

Box No. 2 = 20'

Sh. A-6

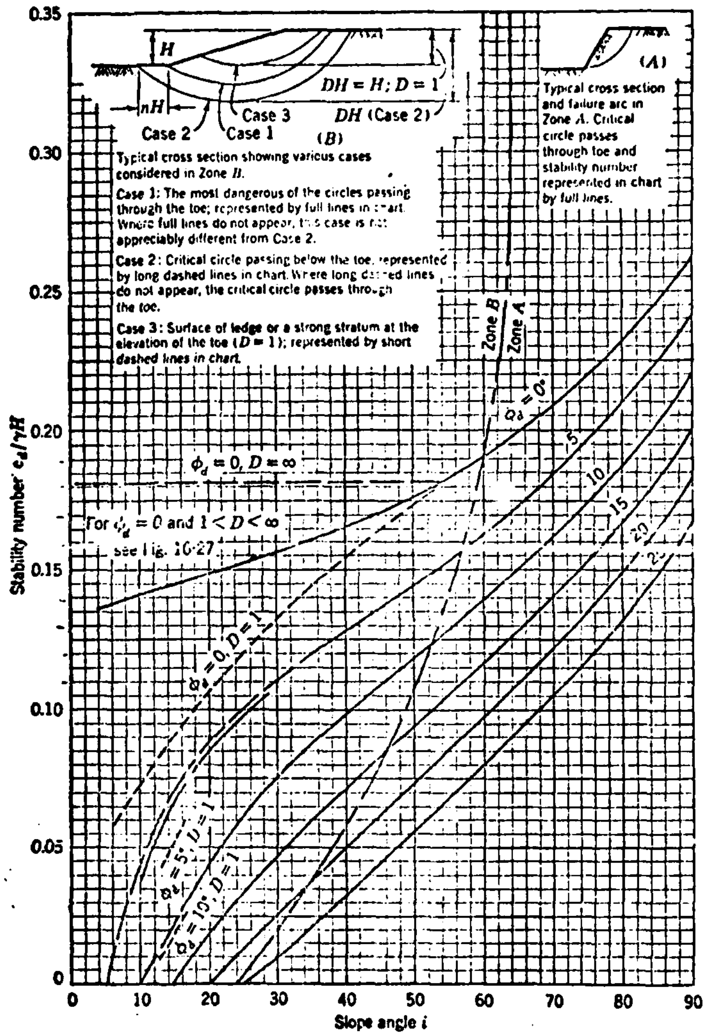


FIG. 16·20 Chart of stability numbers.

$$H_c = \frac{C_d}{\gamma S_N}$$

$$SF = \frac{C_e}{C_d}$$

Ref. D. W. Taylor, "Fundamentals of Soil Mechanics", John Wiley & Sons, Inc. 1948.

Evaluation of Clay Shear Strength

From pocket penetrometer and torvane device used at site on May 1, 1975

1) Coy Glen Sta. 1+50 south bank, stream bed @ Elev 389±

a) T_v = 0.48, 0.48, 0.40, p_p = 1.0, 1.5, 1.5, 1.0 (at water level)

b) T_v = 0.53, 0.57, p_p = 2.0, 2.0, 1.75 (1' above water level)

north bank of stream, 8' above water level

p_p = 1.75, 1.5, 1.5

c) Sta. 1+85 south bank at water level

T_v = 0.4, 0.4, p_p = 1.5, 1.25, 1.5, 1.5

T_v reading = cohesion strength tsf

p_p = unconfined strength (2×C) tsf

Comments

1) T_v tests indicate $c \geq 800$ psf

p_p " " $c \geq 1000$ psf

2) From Sh. A-2 N values @ El. 389 are 3.5 above as at lower levels as N = 2.5 It may be that the shear strength values at this level (El. 389) may be in a desiccated zone.

Adjust ment for N

est. $c \geq \frac{2.5}{3.5} \times 800 = 570$ psf

USE c = 600 psf

Revised Bearing Capacity (Ref. Sh. A-5)

$q = 5.14c + D_f\gamma = 5.14 \times 600 + 13 \times 58 = 3080 + 750 = 3830$ psf

Drop Structure Foundation Load

Dead Load	0.96 ksf		(Str. Calc. Sh. 37A)
DL + Live Load	8.95 "	max.	"
Walls: high wall @ toe	3.87 "		" 44
low " @ "	2.42 "		" 51

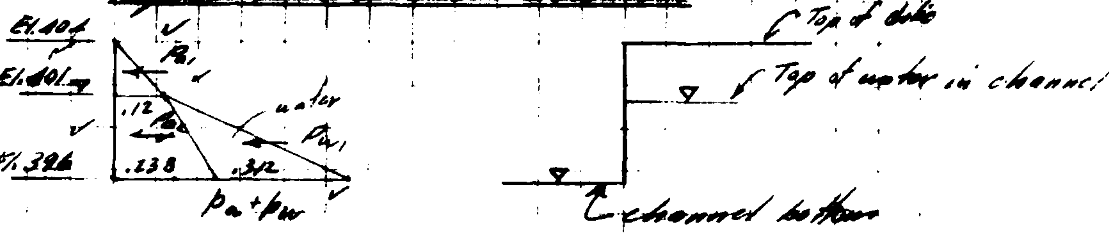
HNTB

MADE BY AAS-J DATE 5-3-75 JOB NO. 4204
CHECKED BY J K B DATE 5-10-75 SEC. NO.
SHEET NO. A-8

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

<u>Check for Sheet Pile Retaining Walls</u>

<u>1. Upstream End of Box Wall</u>



a) <u>For Sand Backfill</u> assume $\phi = 30°$, $K_a = 0.33$*

El. 404 to 401 $\gamma_{moist} = 120$ pcf, $p_a = 0.333 \times 120 = 40$ psf/ft.

El. 401 to 396 $\gamma_{sat.} = 133$ pcf (Sh. A-1), $\gamma_{buoy} = 133 - 62.4 = 70.6$ pcf

$p_a = 0.333 \times 70.6 = 23.5$ psf/ft.

@ El. 401 $p_a = 3 \times 40 = 120$ psf

396 $= 120 + (5 \times 23.5 = 117.5) = 238$ psf ← CONTROLS USE

$p_w = 5 \times 62.4 = 312$

b) <u>For Clay Soil</u> $\gamma_{moist} = \gamma_{sat} = 121$ pcf (Sh. A-1)

$\gamma_{buoy} = 121 - 62.4 = 58.6$ pcf

$C = 600$ psf (Sh. A-7)

$p_a = \gamma h - 2C$*

@ El. 396 $p_a = (3 \times 121 = 363) + (5 \times 58.6 = 293) - (2 \times 600 = 1200)$

$= 656 - 1200 = -544$

c) <u>Resistance Below Elev. 396</u> - Clay Soil

level at which active pressure of clay > 0

$h = 2C/\gamma_b = \dfrac{1200 - \cancel{363} - \cancel{293}}{58.6} = \cancel{14.3}\ 9.3'$ El. $\cancel{381.7}$ 386.7

d). Assume differential water head, El. 401 behind sheeting, E396 in front of sheeting reduces to zero at Elev. 375. sand below Elev. 375 should provide equal water head on each side of sheeting below this level.

Active pressure in clay @ El. 375

$p_a = (3 \times 121 = 363) + (26 \times 58.6 = 1524) - 1200 = 687$ psf

---

* Fig. 10-1 Ref. (1) "Design Manual, Soil Mechanics, Foundations, and Earth Structures, DM-7" U.S. Navy NAVFAC, Mar., 1971

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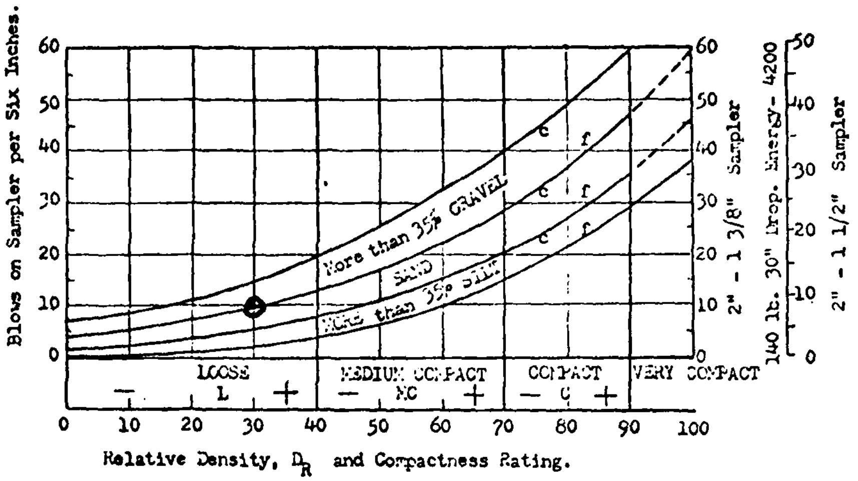


Approximate adjustment of blows per six inches, B' for new weight of hammer, W and height of drop, H and new outside and inside diameters of sampler, D_o and D_i.

New Scale of Blows/6" $$B = B' \times \frac{4200}{W\,H} \times \left[\frac{D_o^2 - D_i^2}{2.0^2 - 1.375^2}\right]$$

FIG. 6.—Compactness Performance Rating for Evaluation of In-Place Relative Densities and Compactness from Boring Records and Blow-Counts on a 2 by 1⅜-In. Sampler under a 140-lb Hammer Falling 30 in. Blow-counts are governed by relative density in the sampling depth and by the influences of coarseness or fineness of soils sampled. (4, Figs. 4 and 5, pp. 1257–1258.)

TABLE III.—COMPACTNESS PERFORMANCE RATING FOR EVALUATION OF BLOW-COUNTS ON A 2 BY 1⅜-IN. SAMPLER UNDER A 140-LB HAMMER FALLING 30 IN.

Relative Density, D_R		0	20	30	40	50	60	70	80		90
		—	LOOSE L	+		—	MEDIUM COMPACT MC	+	—	COMPACT C	+ VERY COMPACT
More than 35% GRAVEL	c	7.3	11.4	14.6	19.2	25.7	32.1	39.0	45.0	51.2	58.5 VC
	f	4.4	7.3	9.4	12.3	17.0	22.8	29.2	33.0	39.5	46.8
SAND	c										
	f	1.5	3.7	5.7	7.9	10.8	14.6	20.5	24.0	29.3	35.0
More than 35% SILT	c										
	f	0.7	1.2	2.3	3.2	5.9	9.4	14.6	18.3	23.5	29.2

(Penetration Resistance in blows/6 inches)

Re: D. M. Burmister, "Physical, Stress-strain and Strength Responses of Granular Soils," ASTM STP No. 322, 1962.

5A. A-1

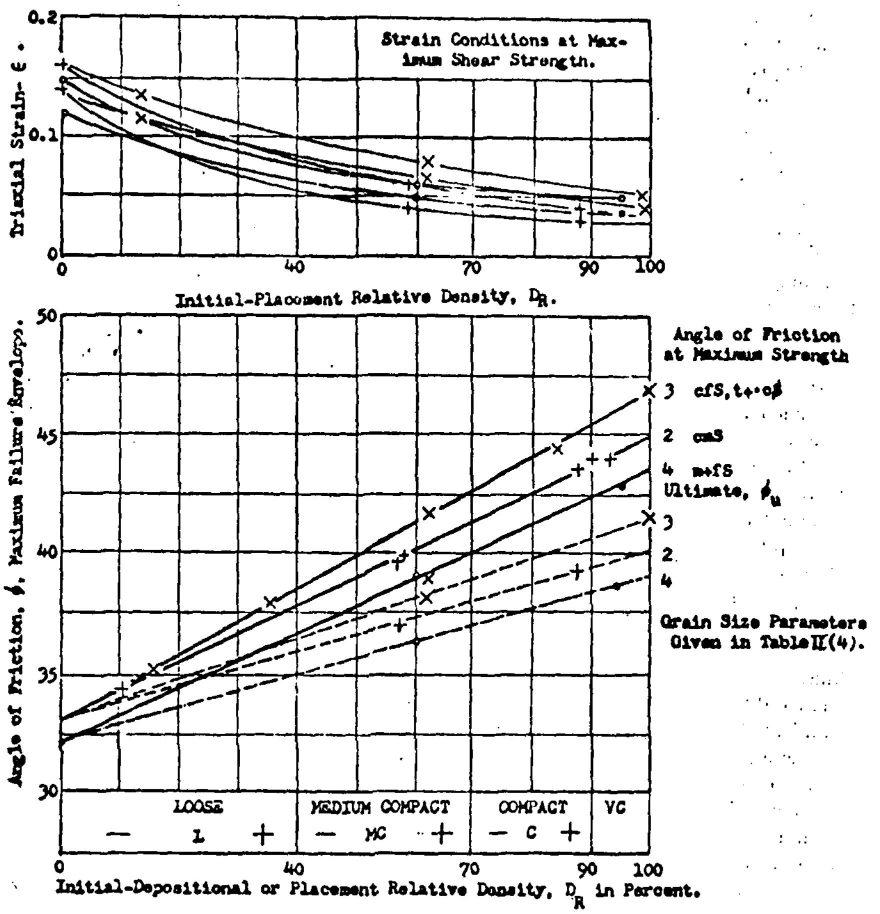


FIG. 14.—Angle of Friction Performance Rating Showing Controlling Influences of Identification and Relative Density of Granular Soils and of Strain Conditions on the Simultaneously Mobilizable Shearing Strengths. (5, Raamot test data.) The angle of friction must be referenced to the initial-depositional relative densities of Fig. 5 as a tentative basis.

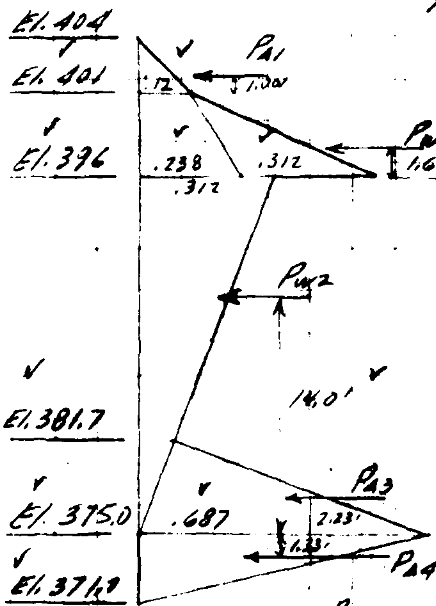
Ref. D. M. Burmister, "Physical, Stress-Strain and Strength Responses of Granular Soils", ASTM STP No. 322, 1962.

e) For Sand and Gravel Layer, El. 375 to 360

av N = 20 (boring SS-D1) Sh. A-2
21 (" SS-D)

For av N = 20, D_r = 30% (Sh. A-9)
for D_r = 30%, min. ϕ = 35° (Sh. A-10)
γ_S = 133 pcf (Sh. A-1)
γ_b = 70.6 " (")

For ϕ = 35° $K_A = \tan^2(45 - \frac{\phi}{2}) = \tan^2(45 - \frac{35}{2}) = \tan^2 27.5° = 0.26$
$K_P = 1/K_A = 3.85$
for S.F. = 1.5* $\frac{K_P - K_A}{1.5}$ = ~~2.4~~ $= \frac{\gamma(K_P - K_A)}{1.5}$ = 70.6 × 2.4
= 170 psf/ft.



P_a	arm	$M_{El.396}$
(.12×5 = 0.60)	2.5 =	1.500
($\frac{1}{2}$×.113×5 = 0.395)	$\frac{5}{3}$ =	.492
.995		1.992

r = 1.992 ÷ .995 = 2.01'

Point p_a = 0
z = .687 ÷ 0.170 = 4.04' El. 371.0

Σ Moments @ Elev. 3710

	P_A & P_W		arm	$M_{El.371.0}$
P_{A1}	($\frac{1}{2}$×3×.12) =	0.180	31.0	5.58
P_{A2}	$\frac{1}{2}$(.12+.238 = .358)5 =	0.896	27.01	24.20
P_{w1}	($\frac{1}{2}$×.312×5) =	0.780	26.67	20.80
P_{w2}	($\frac{1}{2}$×.312×21) =	3.280	18.0	59.10
P_{A3}	($\frac{1}{2}$×.687×6.7) =	2.300	6.23	14.33
P_{A4}	($\frac{1}{2}$×.687×4.04) =	1.388	2.67	3.68
	ΣP_A	8.824	M_o =	127.69 ^k

$\bar{x}$ = 127.69 ÷ 8.824 = 14.48'

Note: Sheets A-12 to A-25 VOID

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Passive pressure in clay - front of sheeting

$p_p = \Sigma\gamma h + \frac{2c}{SF}$ for $c = 0.6$ ksf, SF = 1.5,

@ El. 396 $\Sigma\gamma h = 0$

$p_p = 0 + \frac{2\times0.6}{1.5} = 0.80$ ksf

@ El. 386 $h = 10'$, $\gamma = 0.0586$; $\gamma h = 10\times.0586 = 0.586$

$p_p = 0.586 + .800 = 1.386$ ksf

Point at which active pressure ≥ 0

$h = \frac{2c}{\gamma} = \frac{2\times0.6}{.0586} = 20.5'$ El. 375.5

Passive pressure in clay - behind sheeting

$\Sigma\gamma h$ (for sand above El. 396

@ El. 396 $\Sigma\gamma h = (3\times.120 = .360) + (5\times.0706 = 0.353) = 0.713$

$p_p = \Sigma\gamma h + \frac{2c}{SF} = (0.713 + \frac{2\times0.6}{1.5} = 0.80) = 1.513$ k..

@ El. 386 $p_p = 1.513 + (10\times.0586 = 0.586) = 2.099$ ksf

point at which active pressure > 0

$h = \frac{2c - 0.713}{\gamma} = \frac{(2\times0.6 - 0.713 = 0.487)}{.0586} = 8.3'$ El. 387.7

Water Pressure below El. 396

@ El. 396 $p_w = 5\times62.4 = 312$ psf $= 0.312$

assume $p_w = 0$ at bottom of sheeting (diff. in head from front to back is zero).

Above data plotted on Sh. A-27

		P	arm	M_{396}
P_{a1}	$\frac{1}{2}\times3\times.12 =$	.180	6.00'	1.080
P_{a2}	$0.12\times5 =$	.600	2.50	1.500
	$\frac{1}{2}\times5(.238-.12=.118) =$	.295	1.67	.493
P_w	$\frac{1}{2}\times5\times.0624$	.780	1.67	1.303
$P_1 =$		1.855		4.376

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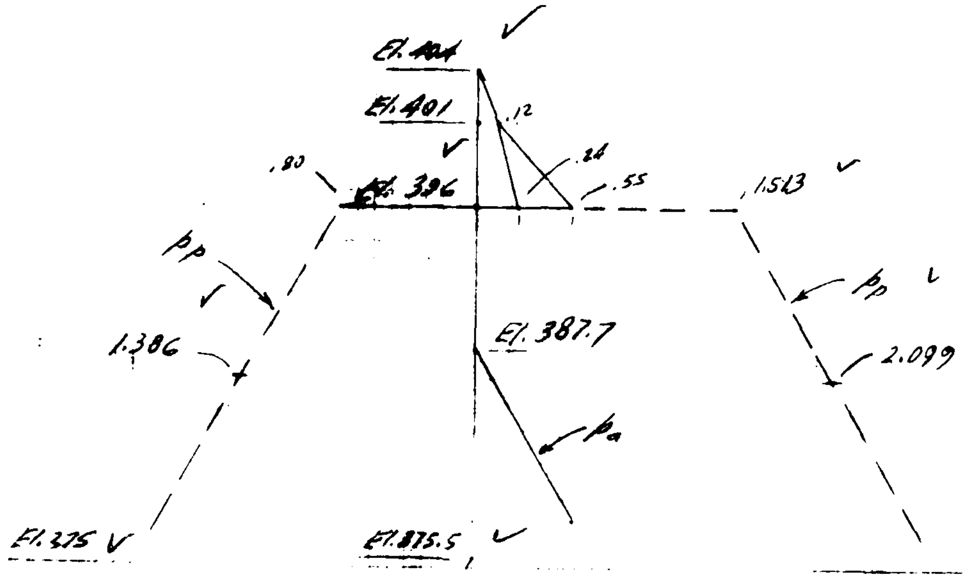
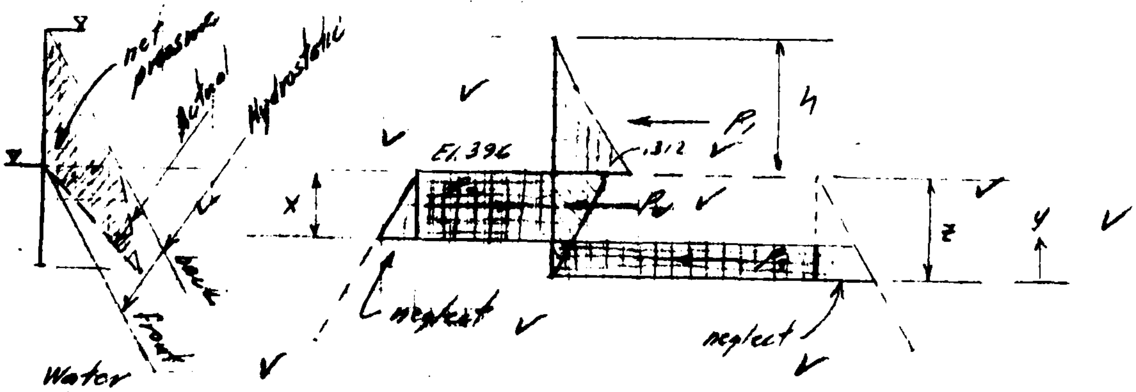
JOB NO. 4024 SHEET NO. A-27
MADE BY AAS-J DATE 6-19-75
CHECKED BY J K J DATE 6.27.75
BACK CHECKED BY ______ DATE ______

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.



$p_p = \gamma h + 2C$ Ref: B.K. Hough "Basic Soils Engineering" Ronald Press, 1957 Eq. 9-23

$p_a = \gamma h - 2C$ Note: use $p_a \geq 0$ (for $2C > \gamma h$, $p_a = 0$)



Water Pressure $\Sigma H = P_1 - P_2 + P_3 + P_w = 0$

$P_2 = x\left(\frac{\gamma x}{2} + 2C\right) \approx x(2C) = 0.80x$
neglect to simplify equations

$P_3 = (\Sigma \gamma h + 2C)(z-y) + \frac{1}{2}\gamma(y^2 - x^2)$

$\approx (\Sigma \gamma h + 2C)(z-y) = 1.513(z-y)$

To simplify analysis assume passive pressure in clay below Elev. 396 is constant with depth - see diagram Sh. A-27

Forces (Ref Sh. A-27)

$P_1 = 1.855^k$

$P_2 = .80x$

$P_3 = 1.513(z - x)$

$P_w = \frac{1}{2}(.312)z = .156z$

$\Sigma H = 0; \quad P_1 - P_2 + P_w + P_3 = 0$

$1.855 - .80x + 1.513z - 1.513x + 0.156z = 0$

$2.313x = 1.855 + 1.669z; \quad x = .802 + .722z$

$x^2 = 0.643 + 1.158z + 0.521z^2$

Moments @ El 396 (Ref Sh. A-27)

$M_1 = -4.376$

$M_2 = -P_2\left(\frac{x}{2}\right) = 0.40x^2$

$M_3 = 1.513z\left(\frac{z}{2}\right) - 1.513x\left(\frac{x}{2}\right) = .757z^2 - .757x^2$

$M_w = P_w\left(\frac{z}{3}\right) = 0.052z^2$

$\Sigma M_o = 0, \quad -M_1 - M_2 + M_3 + M_w = 0$

$-4.376 - 0.40x^2 + 0.757z^2 - 0.757x^2 + 0.052z^2 = 0$

$-4.376 - 1.157x^2 + 0.809z^2 = 0$

$-4.376 - .744 - 1.340z - 0.603z^2 + 0.809z^2 = 0$

$-5.120 - 1.340z + 0.206z^2 = 0$

$z^2 - 6.50z - 24.85 = 0$

$$z = \frac{6.50 \pm (42.25 + 99.40 = 141.65)^{\frac{1}{2}}}{2}$$

$$= \frac{6.50 \pm 11.90}{2} = \frac{18.40}{2} = 9.2$$

Use 9.5' 101

tip Elev. 386.5

HNTB

MADE BY AAS-J DATE 6-20-75 JOB NO. 4204
CHECKED BY J.K.J DATE 6.27.75 SEC. NO.
SHEET NO. A-29

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

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$x = 0.802 + 0.722(9.2) = 0.802 + 6.642 = 7.44'$

Point of zero shear y measured up from bottom of sheeting

$P_3 + P_w - P_2 = 0$

$P_3 = 1.513(z - x) = 1.513(9.2 - 7.44 = 1.76) = 2.663$

$P_w \; \frac{y}{9.2}(0.312)\frac{y}{2} = .017y^2$

$P_2 = 0.80(y - 1.76)$

$2.663 + 0.017y^2 - 0.80y + 1.408 = 0$

$.017y^2 - 0.8y + 4.071 = y^2 - 47.06y + 239.47 = 0$

$y = \frac{+47.06 \pm (2214.64 - 957.88 = 1256.76)^{\frac{1}{2}}}{2} = \frac{47.06 \pm 35.45}{2}$

$y = \frac{11.61}{2} = 5.805'$

Mom @ $y = 5.81'$

$M = P_3[y - (\frac{z-x}{2})] + P_w(\frac{y}{3}) - P_2\frac{(y - z + x)}{2}$

$= 2.663[5.81 - \frac{1.76}{2}] + .017(5.81)^2(\frac{5.81}{3}) -$

$- 0.80(5.81 - 1.76)\frac{(5.81 - 1.76)}{2}$

$= 13.13 + 1.11 - 6.56 = 7.68^{1k}$

for $f_s = 18$ ksi, req'd $SM = \frac{12 \times 7.68}{18} = 5.12$ PMA 22 USE MP115 SM = 5.4

Total Length = 8 + 9.5 = 17.5'

2. Downstream end Box No. 1   PMA 22 USE MP115

Total Length = 12.5'

HNTB

MADE BY AAS-J DATE 6-20-75 JOB NO. 4204
CHECKED BY J.H.J DATE 6.27.75 SEC. NO.
SHEET NO. A-30

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

3. Upstream End of Box No. 2

El. 395.5 - 17.5 = El. 378.0 sheeting above sand layer @ El. 375 PMA 22

USE MP 115 Total Length = 17.5'

4. Downstream End of Box No. 2

El. 386 — Final Ground

El. 383

El. 379 — bottom of channel

El. 375 — bottom of Clay stratum

El. 386, El. 383, El. 379: p_1 .12, p_{a2} .214

For sand backfill above El. 379
@ El. 383 $p_a = 0.12$ (SL-A-8)
@ El. 379 $p_a = 0.12 + 4(.0706).333 = 0.12 + 0.094 = .214$

Note: water level @ El. 383 both sides of sheeting

		P	arm	$M_{El.379}$
P_{A1}	$\frac{1}{2}\times3\times.12$ =	0.180	5.0	0.900
P_{A2}	$4\times.12$ =	0.480	2.0	0.960
	$\frac{1}{2}\times4\times.094$ =	0.188	4/3	0.251
		0.848		2.111

a. Design assuming all clay below El. 379

Passive pressure in clay - front of sheeting @ El. 379
$p_p = 0.80$ ksf (See SL. A-26)
Passive pressure in clay - behind sheeting @ El. 379
$p_p = \Sigma\gamma h + \frac{2c}{SF}$ $\quad \frac{2c}{SF} = \frac{2\times0.6}{1.5} = 0.80$

$\Sigma\gamma h = (3\times.120 = .360) + (4\times.0706 = 0.282) = 0.642$
$p_p = 0.642 + 0.80 = 1.442$ ksf

HNTB

MADE BY AAS-J DATE 6-20-75 JOB NO. 4204
CHECKED BY JAJ DATE 6.27.75 SEC. NO.
SHEET NO. A-31

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

Forces (ref sh. A-27)

$P_1 = 0.848$

$P_2 = -0.8x$

$P_3 = 1.442(z-x)$

$\Sigma F = P_1 + P_3 - P_2 = 0.$

$.848 + 1.442z - 1.442x - 0.8x = 0$

$2.242x = .848 + 1.442z$

$x = 0.378 + 0.643z$; $x^2 = .143 + .486z + .413z^2$

Moments @ El. 379

$M_1 = -2.111$

$M_2 = -P_2 \frac{x}{2} = -0.8x\left(\frac{x}{2}\right) = -0.4x^2$

$M_3 = 1.442z\left(\frac{z}{2}\right) - 1.442x\left(\frac{x}{2}\right) = 0.721z^2 - .721x^2$

$\Sigma M = 0$

$-M_1 - M_2 + M_3 = 0$

$-2.111 - 0.4x^2 + 0.721z^2 - .721x^2 = 0$

$-2.111 - 1.121x^2 + 0.721z^2 = 0$

$-2.111 - .160 - .545z - .463z^2 + .721z^2 = 0$

$-2.271 - .545z + .258z^2 = 0$

$z^2 - 2.11z - 8.80 = 0$

$z = \frac{2.11 \pm (4.45 + 35.2 = 39.65)^{1/2}}{2} = \frac{(2.11 \pm 6.30 = 8.41)}{2}$

$= 4.21'$ depth of clay below El. 379 = 4'±

Say OK

Since it is possible that a differential water head could occur from the back to the front of the sheeting increase depth of embedment to 7'

<u>Use MP-115 (PMA 22)</u> total length = 7'+7' = 14'

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MADE BY AAS-T DATE 6-26-75 JOB NO. 4204
CHECKED BY J.L.T DATE 6-3-75 SEC. NO.
SHEET NO. A-32

CALCULATIONS FOR Coy Glen, Ithaca N.Y. (Deleted ...)

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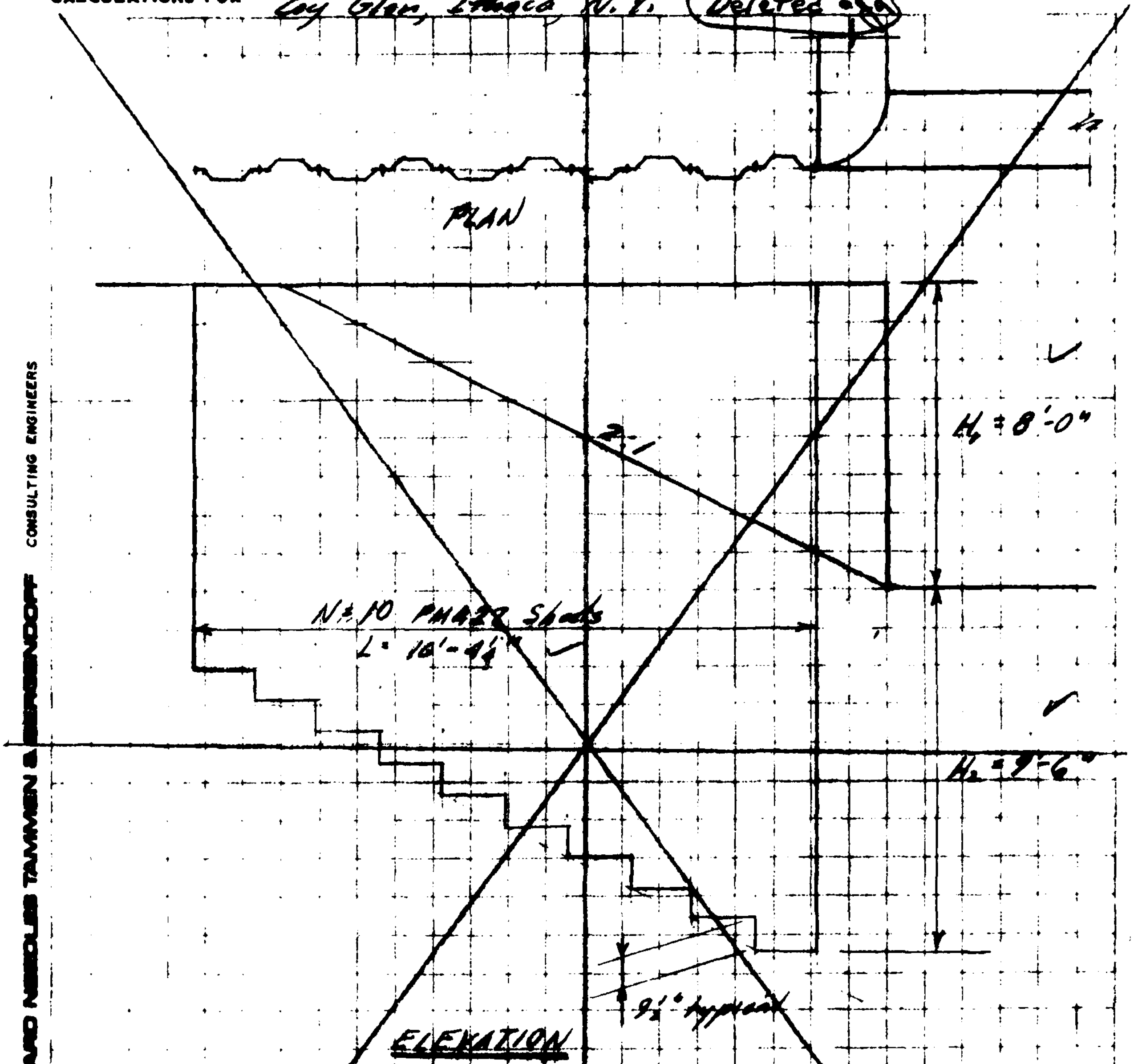
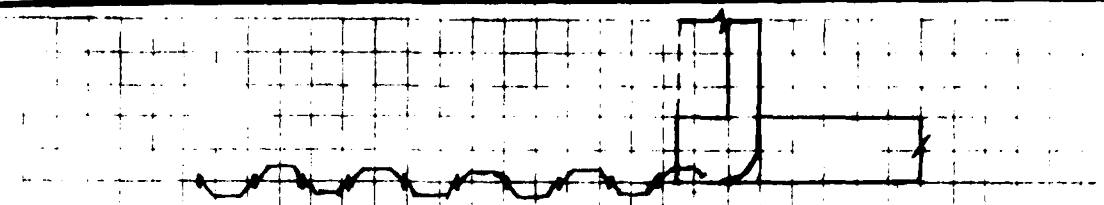


Box No. 1 Upstream & Downstream and Box No. 2 Upstream Wingwalls
Scale 1" = 1'-0"

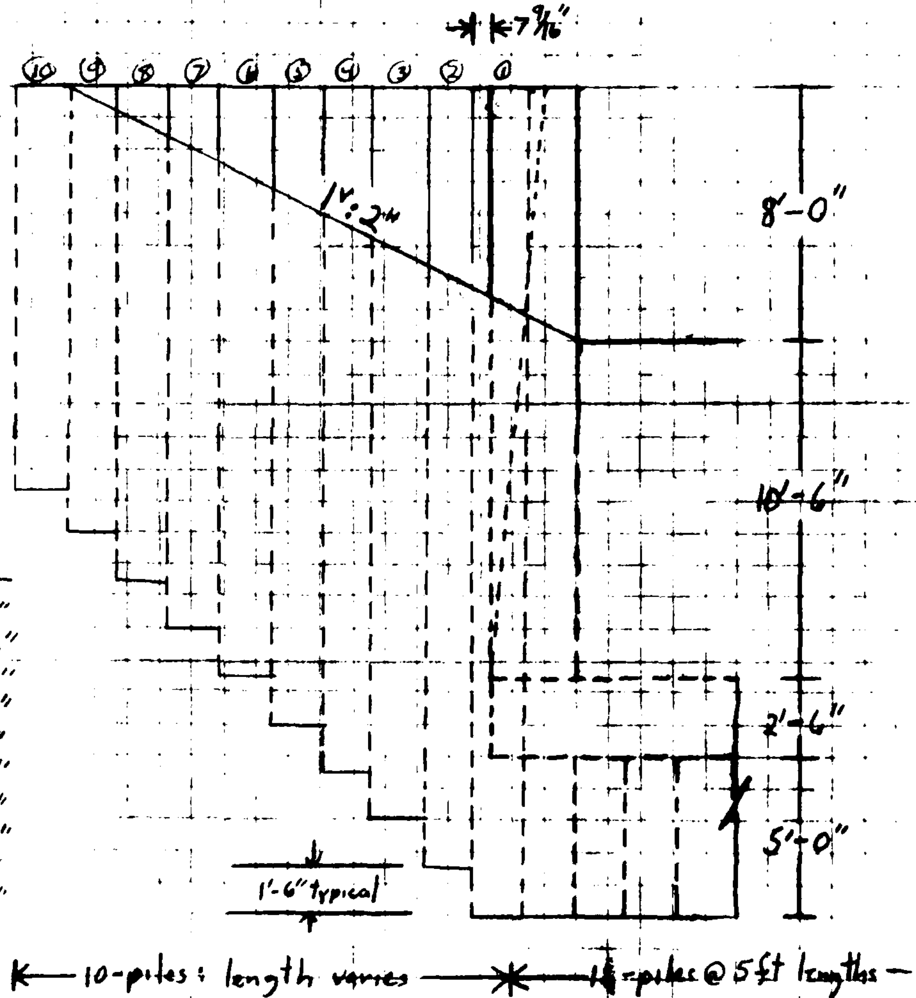
Box No. 2 Downstream Wingwall
Same as above except dimensions
$H_1 = 7'-0"$; $H_2 = 7'-0"$; $N = 9$, $L = 14'-8\frac{5}{8}"$

Page ___ of ___ pages
Sheet A-32A

Subject: Coy Glen, Ithaca, New York
Computation of: Sheet Pile Wingwalls at Upstream End of Drop Structures 1 & 2
Computed by: RJG
Checked by: aga
Date: 18 Dec 1975



PLAN



Pile No	Length
1	26'-0"
2	24'-6"
3	23'-0"
4	21'-6"
5	20'-0"
6	18'-6"
7	17'-0"
8	15'-6"
9	14'-0"
10	12'-6"

ELEVATION

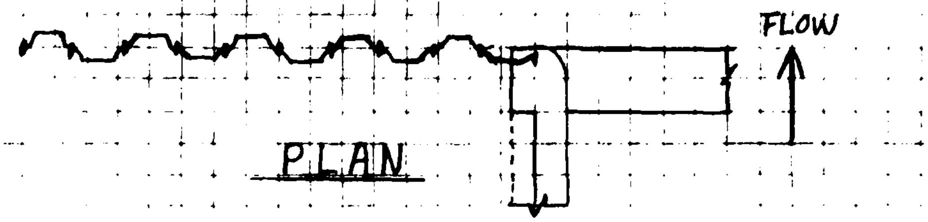
UPSTREAM WINGWALLS FOR

BOX NO. 1 and 2

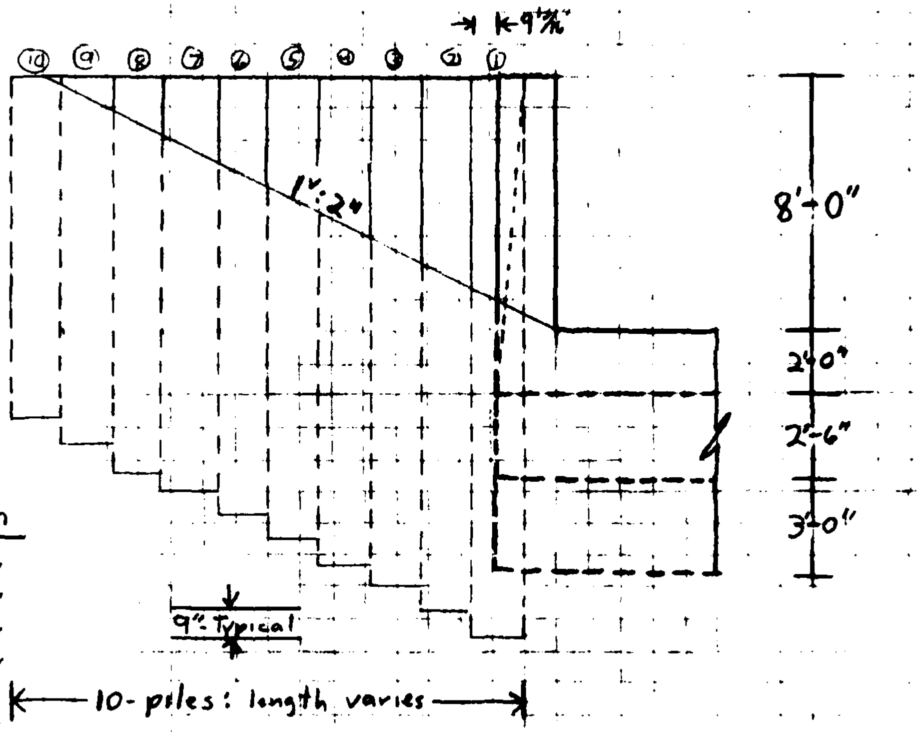
scale: 3/16" = 1'-0"

Page ___ of ___ pages.
Sheet A-32B

Subject: Coy Glen, Ithaca, New York
Computation of: Sheet Pile Wingwalls at Downstream End of Drop Structures 1 & 2
Computed by: RJG   Checked by: [illegible]   Date: 18 Dec 1975



PLAN



Pile No.	Length
1	17'-6"
2	16'-9"
3	16'-0"
4	15'-3"
5	14'-6"
6	13'-9"
7	13'-0"
8	12'-3"
9	11'-6"
10	10'-9"

ELEVATION

DOWNSTREAM WINGWALLS
FOR
BOX NO. 1 and 2

scale: 3/16" = 1'-0"

3. RIPRAP ANALYSIS

3.1 The riprap analysis is for replacement of riprap that has been eroded away in the Cayuga Inlet channel between the Lehigh Railroad bridge and the drop structure.

3.2 An inspection of the site showed that the riprap that could be observed, that on the banks of the channel, was in good condition and exhibited no signs of erosion. Riprap on the bottom of the inlet channel could not be observed because of the depth of water.

3.3 Theoretical analyses of riprap requirements were made of the inlet channel below the railroad bridge, Sheets R-1 to R-6. The theoretical analysis was in good agreement with that specified for the construction, Sheet R-6, Typical measurements of stone on the channel bank, Sheet R-10, showed that the stone placed was reasonably close to that specified for the construction.

3.4 The agreement between the above noted theoretical analysis and performance of the stone on the bank indicate the riprap performance along the channel bank is in agreement with the theoretical design at this site.

3.5 In June 1964, sieve analyses were performed on the foundation material in the area of the proposed riprap repair. An average gradation curve based on these analyses was plotted on ENG Form 2087 (see page 107A). The range for a filter design based on this foundation material and the range for the bedding material specified in the 1967 contract are also

shown on ENG Form 2087. Comparison of the curves indicate that the finer sizes of the bedding gradation specified in the 1967 contract were similar to the native material and the coarser range fell within the filter design limits. Therefore, the existing foundation material and the specified bedding material in the 1967 contract are compatible. ENG Form 2087 also indicates a bedding material which falls within the range of the filter design and is the proposed bedding material to be used in the riprap repair.

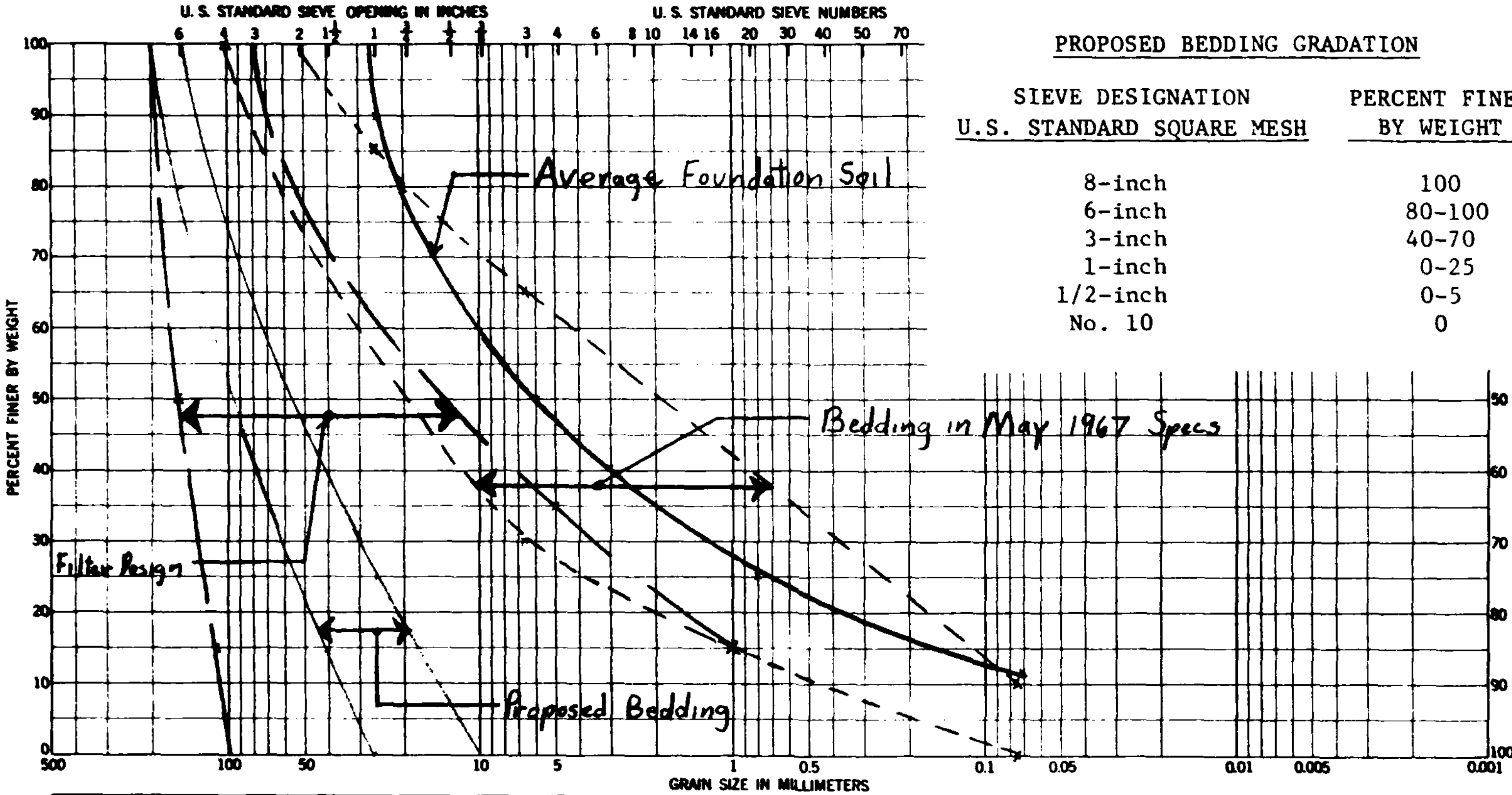
3.6 A theoretical analysis was made for the riprap in the scour area just below the drop structure, Sheets R-7 and 8. It was found that the required size was essentially the same as that used on the original construction. It is concluded that the riprap failure at this location is therefore due to local turbulence occurring because of the drop structure. It appears that the drop structure is of inadequate length for full attenuation of turbulence caused by the water fall.

3.7 Stone sizes for traction shear forces up to 2.8 times the normal value were investigated on Sheet R-9. Since the stone size is a function of the third power of the traction shear force, the resulting stone sizes grew rapidly as the design traction force was increased.

3.8 The stone size proposed by the Buffalo District is about 2.5 times the size theoretically required if turbulence were not present. It also results in an increased traction shear resistance of 25 percent. The use a larger size stone to provide 50 percent increase in traction

PROPOSED BEDDING GRADATION

SIEVE DESIGNATION U.S. STANDARD SQUARE MESH	PERCENT FINER BY WEIGHT
8-inch	100
6-inch	80-100
3-inch	40-70
1-inch	0-25
1/2-inch	0-5
No. 10	0



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project Cayuga Inlet

Area Riprap Repair

Boring No. Foundation & Bedding

Date 10 December 1975

GRADATION CURVES

ENG FORM 1 MAY 63 2087

shear resistance results in a stone size 4 times that theoretically required. The use of any larger size stone is not practical since it would be larger than the scour hole it is to fill.

3.9 It is concluded that the size stone to be used in the scour area will have to be based on judgment since no evaluation of the turbulence present in this area under high flow can be made. The size riprap proposed by the Buffalo District, Sheets R-11 to R-13 are reasonable since it is two and a half times larger than that which was eroded out.

3.10 Riprap designs for the two adjacent Coy Glen drop structures developed by the Buffalo District are included on Sheets R-14 to R-18.

HNTB

MADE BY AAS-J DATE 4-7-75 JOB NO. 4204-99-01
CHECKED BY COV DATE April 11-75 SEC. NO.
SHEET NO. R-1

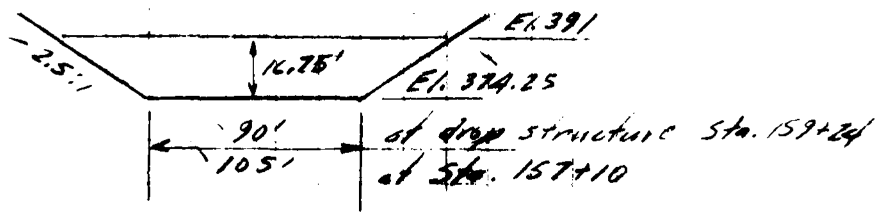
CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Cayuga Inlet Rip-Rap Design Sta. 157+10 & 159+24

Design Flow 16,000 cfs Ref. Inc. F-2
Bottom of Channel El. 374.25 " " F-3, Dwg 238-A-31/5
Top of Water El. 391 " Phone call 4-7-75 to Mr. R. Gorecki DD, CofE
Av. velocity 7.1 to 7.7 ft/sec. " -do-

Channel Section Ref Inc. F-3 (Dwg. 238-A-31/5)



Av. Width $W_1 = 90 + (2.5 \times 16.75 = 41.875) = 131.875'$
$W_2 = 105 + 41.875 = 146.875'$

Area $A_1 = 16.75 \times 131.875 = 2{,}208.9$ s.f.
$A_2 = 16.75 \times 146.875 = 2{,}460.2$ sf.

av. Velocity $V_1 = 16{,}000 \div 2208.9 = 7.24$ ft/sec. vs 7.7
$V_2 = 16{,}000 \div 2460.2 = 6.50$ ft/sec. vs 7.1

Note: Inc. F-3 shows top of Rip Rap @ El. 390.0
For top of water at El. 390.0
Av. Width $W_1 = 90 + (2.5 \times 15.75 = 39.375) = 129.375'$
$A_1 = 15.75 \times 129.375 = 2037.7^{\square'}$
$V_1 = 16{,}000 \div 2037.7 = 7.85$ ft/sec. vs 7.7
$W_2 = 105 + 39.375 = 144.375'$
$A_2 = 15.75 \times 144.375 = 2273.9^{\square'}$
$V_2 = 16{,}000 \div 2273.9 = 7.0$ ft/sec. vs 7.1

USE av velocity = 7.7 & 7.1 ft/sec.
depth = 15.75'

HNTB

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

MADE BY AAS-T DATE 4-8-75 JOB NO. 4204
CHECKED BY COV DATE April 11-75 SEC. NO.
SHEET NO. R-2

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

<u>Local Boundary Shear</u> Ref EM 1110-2-1601, Eq. 32

$$\tau_0 = \gamma\left[\frac{v}{32.6\log_{10}\frac{12.2y}{D_{50}}}\right]^2$$

v = velocity = 7.7 fps (more critical than $v = 7.1$)
y = channel depth = 15.75'
D_{50} = av. stone diameter – try 1.0'
γ = unit wt. of water = 62.4 pcf

$$\frac{12.2y}{D_{50}} = \frac{12.2\times15.75}{1.0} = 192;\quad \log_{10}\frac{12.2y}{D_{50}} = 2.283$$

$$32.6\times\log_{10}(\ ) = 32.6\times2.283 = 74.3$$

$$v\div32.6\log(\ ) = 7.7\div74.3 = 0.1035$$

$$[v\div32.6()]^2 = 0.1035^2 = 0.0107$$

$$\tau_0 = 62.4[\ ]^2 = 62.4\times0.0107 = 0.67\text{ psf}$$

τ_{bmax}/τ_0 = 2.0 for smooth channel EM 1110-2-1601 Pl 33
= 2.8 " rough " "

USE $\tau_{bmax}/\tau_0 = 2.8$

$$\tau_{b_{max}} = 2.8\times0.67 = 1.88\text{ psf}$$

Design $\tau_b = 1.5\,\tau_{bmax} = 1.5\times1.88 = 2.82$ psf Ref. ETL 1110-2-120 3.d(4)

For side slope of 1V on 2.5H channel

$$\tau' = \tau\left(1-\frac{\sin^2\phi}{\sin^2\theta}\right)^{\frac{1}{2}}$$ Ref EM 1110-2-1601 Eq. 34

$\phi = \tan^{-1}\frac{1}{2.5} = \tan^{-1}0.4 = 21.8°$ $\sin 21.8° = .371$
$\sin^2 21.8 = 0.138$

$\theta = 40°$ $\sin 40° = 0.642$, $\sin^2 40° = 0.413$

$$\tau' = 2.82\left[1-\left(\frac{0.138}{0.413} = 0.335\right) = 0.665\right]^{\frac{1}{2}} = 2.82\times0.815 = 2.30\text{ psf}$$

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Req'd Shear Resistance of Rip-Rap

Channel bottom $\tau = 2.82$

" side $\tau' = 2.30$

$$D_{50} = \frac{\tau}{a(\gamma_s - \gamma)}$$ Ref. EM 1110-2-1601 Eq. 33

$a = 0.040$

γ_s = unit wt. of stone

Limestone γ = 159 to 166 Ref: Table 2.1, Krynine & Judd "Principles of Engrg Geology and Geotechnics"

Sandstone 119 to 161

USE $\gamma_s = 160$ pcf

$\gamma = 62.4$

Req'd min. $D_{50} = \frac{2.82}{0.04(160-62.4)} = \frac{2.82}{(0.04 \times 97.6 = 3.90)} = 0.72'$

Req'd $V = \frac{\pi D^3}{6} = \frac{\pi}{6}(0.72)^3 = \frac{\pi}{6} 0.373 = 0.195$ cf.

Req'd min. $W_{50} = 0.195 \times 160 = 31.2$ lbs.

For $\gamma_s = 160$ pcf, Thickness = 24" min. $W_{50} = 40$ lbs.
Ref. ETL 1110-2-120 Incl 2, p 2 *

A Check for $\gamma_s = 155$ pcf

Req'd min $D_{50} = \frac{2.82}{0.04(155-62.4 = 92.6)} = 0.76'$

Req'd $V = \frac{\pi}{6}(0.76^3 = 0.44) = 0.230$ cf

Req'd min. $W_{50} = 0.23 \times 155 = 35.6$ lb.

Note for $\gamma = 155$ 15" thickness min $D_{50} = 38^{\#}$ (Ref ETL 1110-2-120, Incl. 2, p2)*
18" " $55^{\#}$

Check τ_0 for min. $D_{50} = 40^{\#}$ $\gamma = 160$ pcf
$= 38^{\#}$ & $55^{\#}$ for $\gamma = 155$ psf

* Assumed to be placed under water.

HNTB

MADE BY ______ DATE ______ JOB NO. ______
CHECKED BY COO DATE April 11-75 SEC. NO. ______
SHEET NO. R-4

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

(1) For min. D_{50} = 40#, γ = 160 pcf

Design shear Force, $\tau_{b\,max}$

Vol. = 40 ÷ 160 = 0.250 ft³

$$D = \left(\frac{6V}{\pi}\right)^{\frac{1}{3}} = \left(\frac{6 \times 0.250}{\pi} = 0.477\right)^{\frac{1}{3}} = 0.781'$$

calc. τ_o $\quad \frac{12.2y}{D_{50}} = \frac{12.2 \times 15.75}{0.781} = 246$

$$\log_{10} \frac{12.2y}{D_{50}} = 2.390$$

$$32.6 \log_{10}(\;) = 32.6 \times 2.390 = 78.0$$

$$V \div 32.6 \log_{10}(\;) = 7.7 \div 78.0 = 0.0986$$

$$[V \div 32.6 \log_{10}(\;)]^2 = 0.0986^2 = 0.0097.0$$

$$\tau_o = 62.4 \times 0.0097 = 0.606 \text{ psf}$$

Design $\tau_{b\,max}$ = 0.606 × 2.8 × 1.5 = 2.54 psf

design $\tau_{b\,max}$ = 1.5 $\tau_{b\,max}$ (Sh. R-2)

for rough channel $\tau_{b\,max}/\tau_o$ = 2.8 (Sh. R-2)

Inplace shear resistance, τ & τ'

(a) bottom: $\tau = a(\gamma_s - \gamma) D_{50} = 0.04(160 - 62.4 = 97.6)\,0.781$
$= 3.05 > 2.54$ channel bottom OK

(b) channel side

$$\left(1 - \frac{\sin^2\phi}{\sin^2\theta}\right)^{\frac{1}{2}} = 0.815 \text{ (Sh. R-2)}$$

$\tau' = 3.05 \times 0.815 = 2.48 \approx 2.54$ channel sides - Say OK

Conclusion: min D_{50} = 40#, γ = 160 psf OK

2.48 ÷ 2.54 = 0.98 2% low

(2) For min. D_{50} = 38#, γ = 155 pcf

Design Shear Force $\tau_{b\,max}$.

Vol = 38 ÷ 155 = 0.245 ft³, from (1) above then

D = 0.777' and design $\tau_{b\,max}$ = 2.54

Inplace Shear Resistance; τ_b & τ_s

(a) bottom: τ = 0.040(155 - 62.4 = 92.6) 0.777 = 2.88 > 2.54 OK

(b) side τ' = 2.88 × 0.815 = ~~2.43~~ 2.34 < 2.54 NG

Conclusion: min. D_{50} = 32#, γ = 155 pcf NG

2.43 ÷ 2.54 = 0.9[illegible], [illegible] low

HNTB

MADE BY AAS-J DATE 4-8-75 JOB NO. 4204
CHECKED BY [illegible] DATE april 11-75 SEC. NO.
SHEET NO. R-5

CALCULATIONS FOR Cay Glass, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

3) For min. D_{50} = 55lb, γ = 155 pcf

Design Shear Force, $\tau_{b\,max}$

Vol. = 55 ÷ 155 = 0.354 c.f.

$$D = \left(\frac{6 \times 0.354}{\pi}\right)^{1/3} = 0.677^{1/3} = 0.877'$$

Calc. τ_o $\quad \frac{12.2y}{D_{50}} = \frac{12.2 \times 15.75}{0.877} = 219$

$$\log_{10} \frac{12.2y}{D_{50}} = 2.34$$

$$32.6 \log_{10}(\ ) = 32.6 \times 2.34 = 76.2$$

$$V \div 32.6 \log_{10}(\ ) = 7.7 \div 76.2 = 0.101$$

$$[V \div 32.6 \log_{10}(\ )]^2 = 0.101^2 = 0.0102$$

τ_o = 62.4 × 0.0102 = 0.64 psf

Design $\tau_{b\,max}$ = 0.64 × 2.8 × 1.5 = 2.69 psf

Inplace Shear Resistence, τ & τ'

(a) bottom $\tau = a(\gamma_s - \gamma) D_{50}$ = 0.040 (155 − 62.4 = 92.6) 0.877

= 3.25 psf > 2.69 ok

(b) side τ' = 3.25 × 0.815 = 2.65 ≈ 2.69 say Ok

Ref: ETL 1110-2-120 3.c.(5)

2.65 ÷ 2.69 = 0.98 2% low

(4) Check for γ = 150 pcf per R. Gorecki by phone 7 Apr. 75

Try min D_{50} = 73 lbs Ref. ETL 1110-2-120 Incl. 2, p1

Design Shear Force; $\tau_{b\,max}$

Vol. = 73 ÷ 150 = 0.487 c.f.

$$D = \left(\frac{6 \times 0.487}{\pi}\right)^{1/3} = 0.93^{1/3} = 0.976'$$

Calc. τ_o $\quad \frac{12.2y}{D_{50}} = \frac{12.2 \times 15.75}{0.976} = 197$

$$\log \frac{12.2y}{D_{50}} = 2.294$$

$$32.6 \log_{10}(\ ) = 32.6 \times 2.294 = 75.0$$

$$V \div 32.6 \log_{10}(\ ) = 7.7 \div 75 = 0.1025$$

$$[V \div 32.6 \log_{10}(\ )]^2 = 0.0105$$

τ_o = 62.4 × 0.0105 = 0.655 psf

H2

HNTB

MADE BY AAS-T DATE 4-8-75 JOB NO. 4204
CHECKED BY CUU DATE April 11-75 SEC. NO.
SHEET NO. R-6

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

Design $\tau_{b\,max}$ = 0.655 × 2.8 × 1.5 = 2.75 psf

Inplace Shear Resistance τ & τ'

(a) bottom $\tau = a(\gamma_s - \gamma)D_{50}$ = 0.04 (150-62.4=87.6) 0.976
= 3.42 psf ≥ 2.75 ok

(b) side τ' = 3.42 × 0.815 = 2.79 psf ≥ 2.75 ok
2.75 ÷ 2.79 = 0.98 2% low

Summary

1) for unit wt. of stone, γ_s = 160 pcf
min D_{50} = 40 lbs Ref. Sh. R-4

Percent Passing	Stone Size	Ref. ETL 1110-2-120 Incl. 2, p2
100	199 - 79	
50	59 - 40	
15	29 - 12	

3) for unit wt. of stone, γ_s = 155 pcf Ref - same
min. D_{50} = 55 lbs Ref. Sh. R-5

100	274 - 110
50	81 - 55
15	41 - 17

4) for unit wt. of stone, γ_s = 150 pcf Ref same
min. D_{50} = 73 pcf Ref. Sh. R-6 p1

100	364 - 145
50	108 - 73
15	54 - 23

Note: Corps of Engineer Specs for Existing Cayuga Inlet Construction Specs for Rip-Rap (Mr. R. Gorecki by phone May 2, 1975)

D_{100} 250 lb.
D_{50} 65 lb.
D_{15} 15 lb.

HNTB MADE BY AAS-J DATE 4-21-75 JOB NO. 4204-99-01
CHECKED BY SWE DATE 5/7/75 SEC. NO.
SHEET NO. R-7

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Channel Section @ Sta. 160+00

Check velocity
Width 80' (Ref. Enc. F-7)
Bottom Elev 374.28 (" " F-3, Dwg 238-A-31/5)
Water surface El. 390 (top of rip-rap Enc. F-3)
$y = 15.72'$

$Area = (390 - 374.28 = 15.72)\,80 = 1257.6^{\square'}$

Design Flood 16,000 cfs (Enc. F-2)

$Av.\ Vel. = 16{,}000 \div 1257.6 = 12.7$ ft/sec.

Local Boundary Shear. Ref. EM 1110-2-1601, Eq. 3c
Assume $D_{50} = 1.0'$

$$\tau_0 = \gamma \left(\frac{V}{32.6 \log_{10} \frac{12.2y}{D_{50}}} \right)^2$$

$$\frac{12.2y}{D_{50}} = \frac{12.2 \times 15.7}{1} = 191.5 \; ; \; \log \frac{12.2y}{D_{50}} = 2.282$$

$$32.6 \log_{10} [\;] = 32.6 \times 2.282 = 74.4$$

$$\tau_0 = 62.4 \left(\frac{12.7}{74.4} = 0.171 \right)^2 = 62.4 \times .03 = 1.87$$

1) Design τ_0 use $1.5\,\tau_0$ Ref: ETL 1110-2-120 Sec. 3.d(4)
$= 1.5 \times 1.87 = 2.80$ psf vs 2.82* sh. R-2
* based on design $\tau_0 = 1.5 \times 2.8\,\tau_0$ where 2.8 factor is for conditions of bend in channel.

2) Since turbulance is likely to exist at this location and it could be on the order of that encountered at a bend (Ref. R 33, EM 1110-2-1601) also check Design $\tau_0 = 2.8\,\tau_0 = 2.8 \times 2.8 = 7.84$ psf

HNTB

MADE BY AAS-J DATE 4-21-75 JOB NO. 4204
CHECKED BY SWE DATE 5/7/75 SEC. NO.
SHEET NO. R-8

CALCULATIONS FOR Coy Glen Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

For $\gamma = 150$ pcf

1) For design $\tau_o = 2.8$ psf

Req'd $D_{50} = \frac{\tau}{a(\gamma_s - \gamma)} = \frac{2.8}{0.04(150-62.4)} = \frac{2.8}{0.04 \times 87.6} = 0.8'$

Req'd volume $= \frac{\pi D^3}{6} = 0.27$ cf ; req'd wt $= 0.27 \times 150 = 40^{\#}$

From ETL 1110-2-120, Incl. 2, page 1 for $\gamma = 150$ pcf
Min. $D_{50} = 53$ pcf

$Vol = 53 \div 150 = 0.354$ c.f.
req'd $D^3 = \frac{6V}{\pi} = \frac{6 \times .354}{\pi} = .676$
$D = (.676)^{\frac{1}{3}} = .878$ ft.

Design Shear, τ_o', for $D = 0.88'$
$\frac{12.2y}{D_{50}} = \frac{12.2 \times 15.72}{0.88} = 218$; $\log_{10} \frac{12.2y}{D_{50}} = 2.338$

$32.6 \log_{10} \frac{12.2y}{D_{50}} = 32.6 \times 2.338 = 76.3$

$\tau_o = 62.4\left(\frac{12.7}{76.3} = 0.166\right)^2 = 62.4 \times .0278 = 1.73$ psf

Design $\tau_o' = 1.5\tau^o = 1.5 \times 1.73 = 2.59$ psf

Shear Resistance ; $\tau = D_{50}\, a\, (\gamma - \gamma_s)$
$= 0.88 \times 0.04 (150 - 62.4 = 87.6)$
$= 3.08$ psf > 2.59 OK

Min $D_{50} = 53^{\#}$

From Sh. R-6 existing $D_{50} = 65$ lb > 53 lb.
Since existing rip-rap, which has eroded, is escentially equal to theoretical, turbulence must exist or a shear force $\tau_o > 2.8$ should be checked.

HNTB

MADE BY AAS-Y DATE 5-6-75 JOB NO. 4604
CHECKED BY SWE DATE 5/7/75 SEC. NO.
SHEET NO. R-9

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

Check req'd D_{50} for τ_o = 1.25, 1.5, 2.0 & 2.8 × 2.8

2) For $\tau_o = 2.8 \times 2.8 = 7.84$ psf

$$\text{Req'd } D_{50} \frac{\tau}{a(\gamma_s - \gamma)} = \frac{7.84}{0.04 \times 87.6} = 2.24'$$

$$\text{Req'd Volume} = \frac{\pi D^3}{6} = \frac{\pi}{6}(2.24)^3 = 5.88 \text{ cf}$$

@ 150 pcf, W_{50} = 882 lbs

~~Req'd size appears too large~~

3) For $\tau_o = 1.5 \times 2.8 = 4.2$ psf

$$\text{Req'd } D_{50} = \frac{4.2}{0.04 \times 87.6} = 1.20'$$

$$\text{Req'd Vol.} = \frac{\pi}{6}(1.20)^3 = 0.90 \text{ cf}$$ @ 150 pcf; W_{50} = 135 lbs

4) For $\tau_o = 2.0 \times 2.8 = 5.6$ psf

$$\text{Req'd } D_{50} = \frac{5.6}{0.04 \times 87.6} = 1.60'$$

$$\text{Req'd Vol.} = \frac{\pi}{6}(1.60)^3 = 2.14 \text{ c.f.}$$ @ 150 pcf W_{50} = 321 lb.

5) For $\tau_o = 1.25 \times 2.8 = 3.5$ psf

$$\text{Req'd } D_{50} = \frac{3.5}{0.04 \times 87.6} = 1.00'$$

$$\text{Req'd Vol.} = \frac{\pi}{6}(1.00)^3 = 0.52 \text{ c.f.}$$ @ 150 psf W_{50} = 78 lbs.

Summary

No.	1)	5)	3)	4)	2)	C of E
Design τ_o	2.8	3.5	4.2	5.6	7.84	Design
min. W_{50}	53#	78#	135#	321#	882#	
ETL 1110-2-120 Incl. 2 Recommendations						
W_{100}	265-106	484-194	998-399	2121-848	-	700-250
W_{50}	79-53	143-97	296-200	628-424	-	250-135
W_{15}	39-17	72-30	148-62	314-133	-	100-40
T	27"	33"	42"	54"		N6

HNTB MADE BY AAS-J DATE 5-2-75 JOB NO. 4204
CHECKED BY ______ DATE ______ SEC. NO. ______
SHEET NO. R-10

CALCULATIONS FOR Coy Glen, Ithaca, N.Y.

HOWARD NEEDLES TAMMEN & BERGENDOFF CONSULTING ENGINEERS

<u>Observed Rip-Rap</u> - Cayuga Channel below drop structure, south bank

Largest Stone
1.0 × 1.3 × 1.7 = 2.2 c.f. @ 150 pcf = 330 lb
2.0 × 1.1 × 1.6 = 3.5 " 525 lb
1.4 × 1.5 × 1.6 = 3.35 " = 500 lb

Median Stone
0.6 × 0.6 × 0.8 = 0.29 c.f. @ 150 pcf = 44 lb
0.6 × 0.6 × 0.9 = 0.32 " 48 lb
0.6 × 0.7 × 0.7 = 0.29 " 44 lb

RIPRAP REPAIR
for
CAYUGA INLET

1. The stone for riprap shall be placed in a layer 24 inches thick and shall conform to the following gradation and as shown on Plate 1 attached to this Inclosure:

Riprap Gradation

% Lighter by Weight	Limits of Stone, Weight in Pounds Maximum	Minimum
100	700	250
50	250	135
15	100	40

2. Where sufficient material has been eroded to require underlayers, the following are recommended:

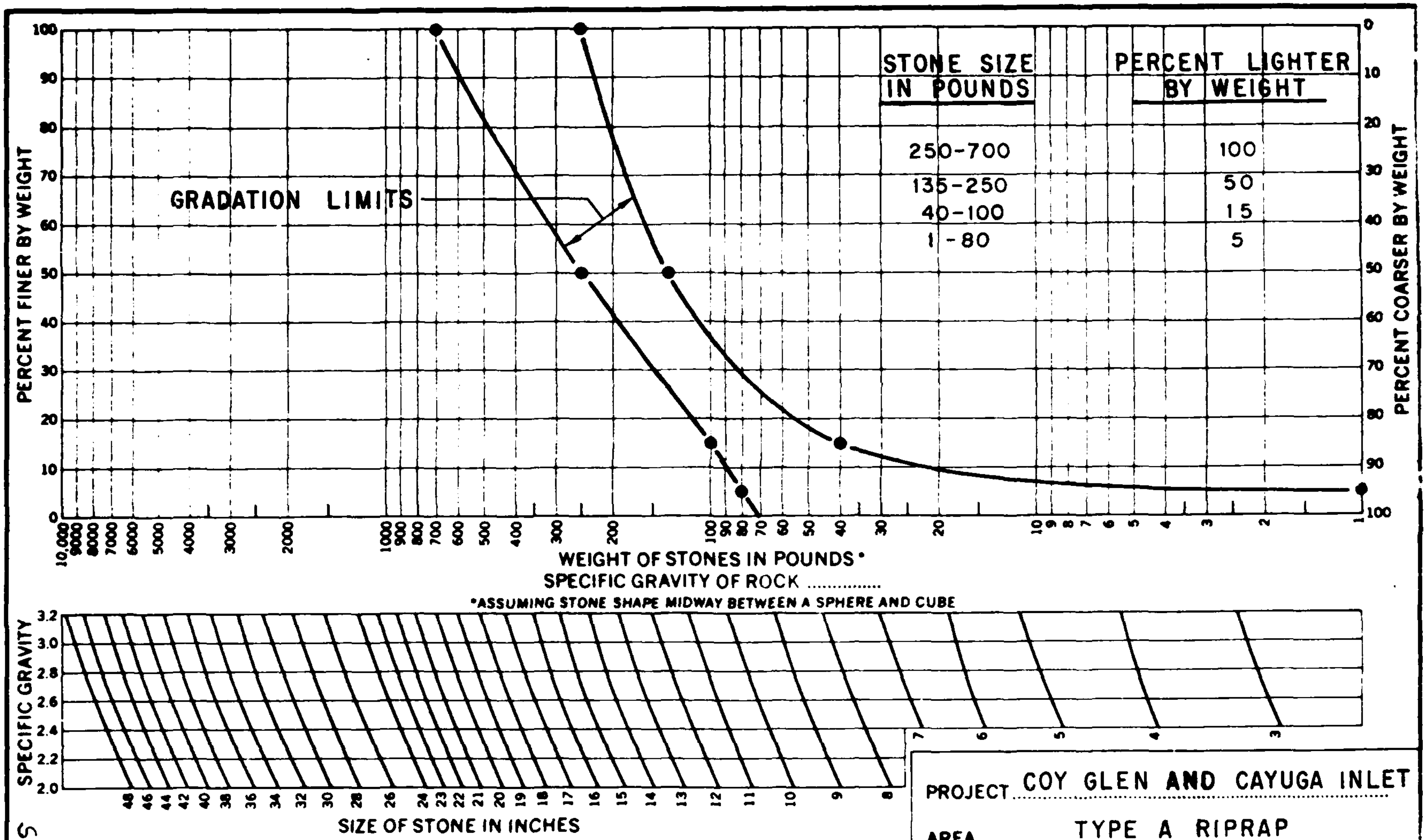
a. Spalls - see gradation below and Plate 2 attached to this inclosure

b. Sand and/or gravel - similar to a concrete aggregate mix

Spalls Gradation

U.S. Standard Sieve Size (inches)	Percent Passing By Weight
8	100
6	80-100
3	40-70
1	0-25
1/2	0-10

Sh. R-11
118



PROJECT COY GLEN AND CAYUGA INLET

AREA TYPE A RIPRAP

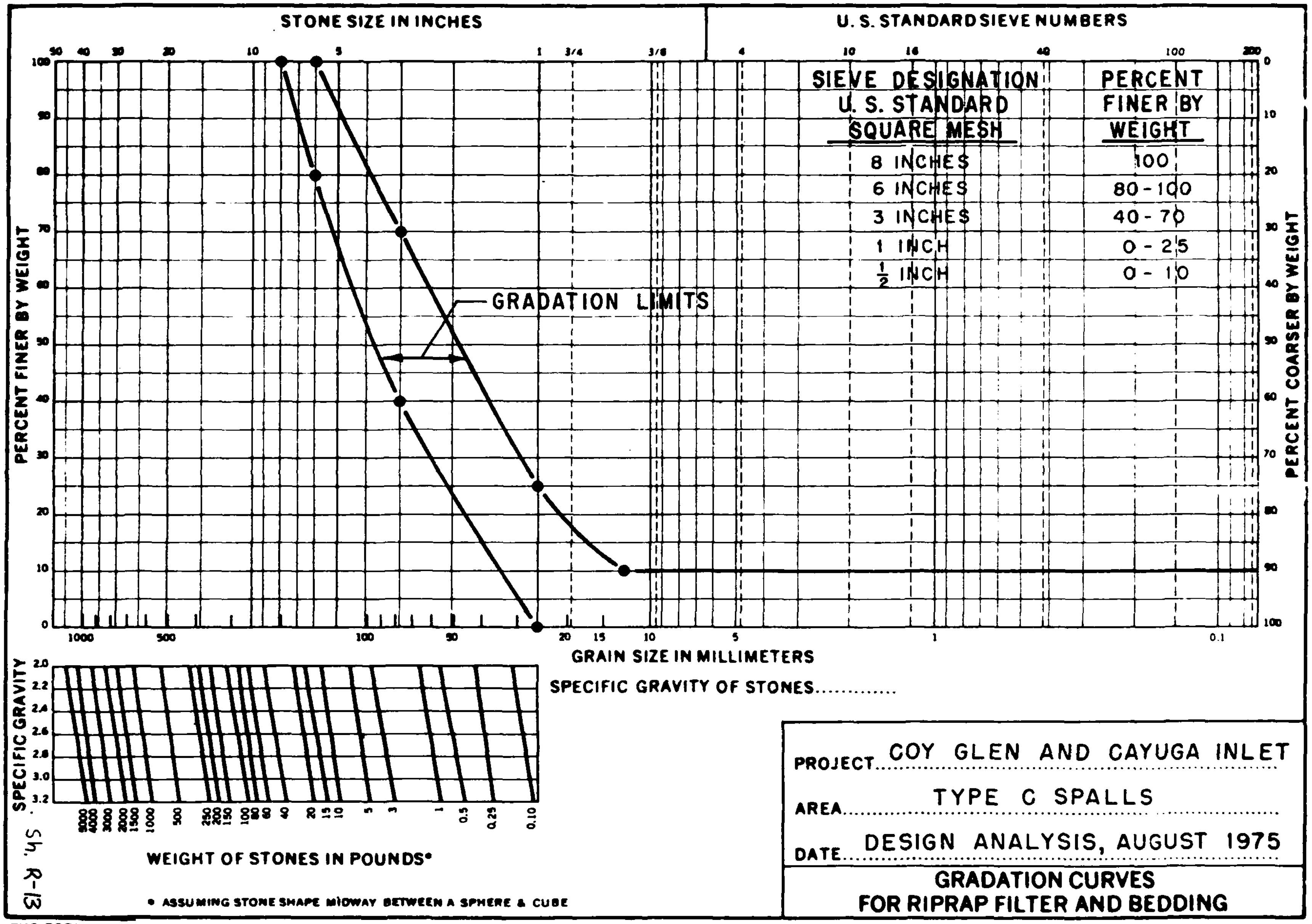
DATE DESIGN ANALYSIS, AUGUST 1975

RIPRAP GRADATION CURVES

ENG FORM 4055 APR 67

PLATE 1

Sh. R-12



SIEVE DESIGNATION U. S. STANDARD SQUARE MESH	PERCENT FINER BY WEIGHT
8 INCHES	100
6 INCHES	80-100
3 INCHES	40-70
1 INCH	0-25
$\frac{1}{2}$ INCH	0-10

PROJECT COY GLEN AND CAYUGA INLET

AREA TYPE C SPALLS

DATE DESIGN ANALYSIS, AUGUST 1975

GRADATION CURVES FOR RIPRAP FILTER AND BEDDING

ENG FORM 4056 APR 67

PLATE 2

RIPRAP DESIGN
FOR
COY GLEN

1. The riprap to be placed 25 feet upstream and downstream of each drop structure and in the channel section from stations 0+00 to station 0+25 shall be in a layer 24 inches thick with 9 inches of bedding and shall conform to the following gradations and as shown on plates 1 and 2 immediately following.

Riprap Gradation

% by Weight Passing	Limits of Stone Weight in Pounds Maximum	Minimum
100	700	250
50	250	135
15	100	40

Bedding Gradation

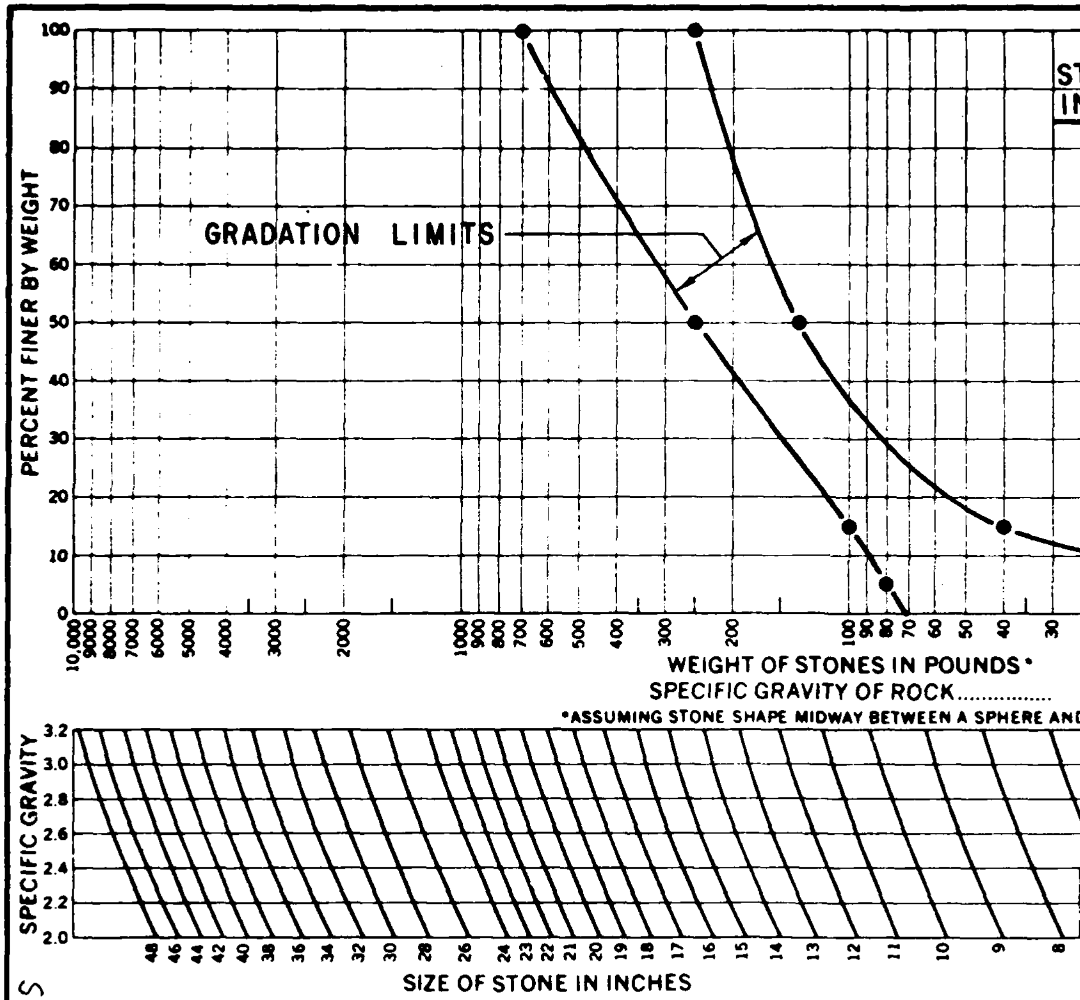
U. S. Standard Sieve Size (inches)	Percent Finer By Weight
4	100
2	65-100
1	50-90
3/4	45-83
No. 4	25-60
10	14-48
40	0-30
No. 200	0-10

Sh. R-14

2. In the areas between station 0+25 to station 1+75 and stations 2+53 to station 3+03, the riprap shall be placed in a layer 21 inches thick with 9 inches of bedding. The riprap shall conform to the following gradation and as shown on plate 3; the bedding material shall conform to the gradation above and as shown on plate 2 immediately following.

Riprap Gradation

% by Weight Passing	Limits of Stone Weight in Pounds Maximum	Minimum
100	300	110
50	150	60
15	50	15



PROJECT COY GLEN AND CAYUGA INLET

AREA TYPE A RIPRAP

DATE DESIGN ANALYSIS, AUGUST 1975

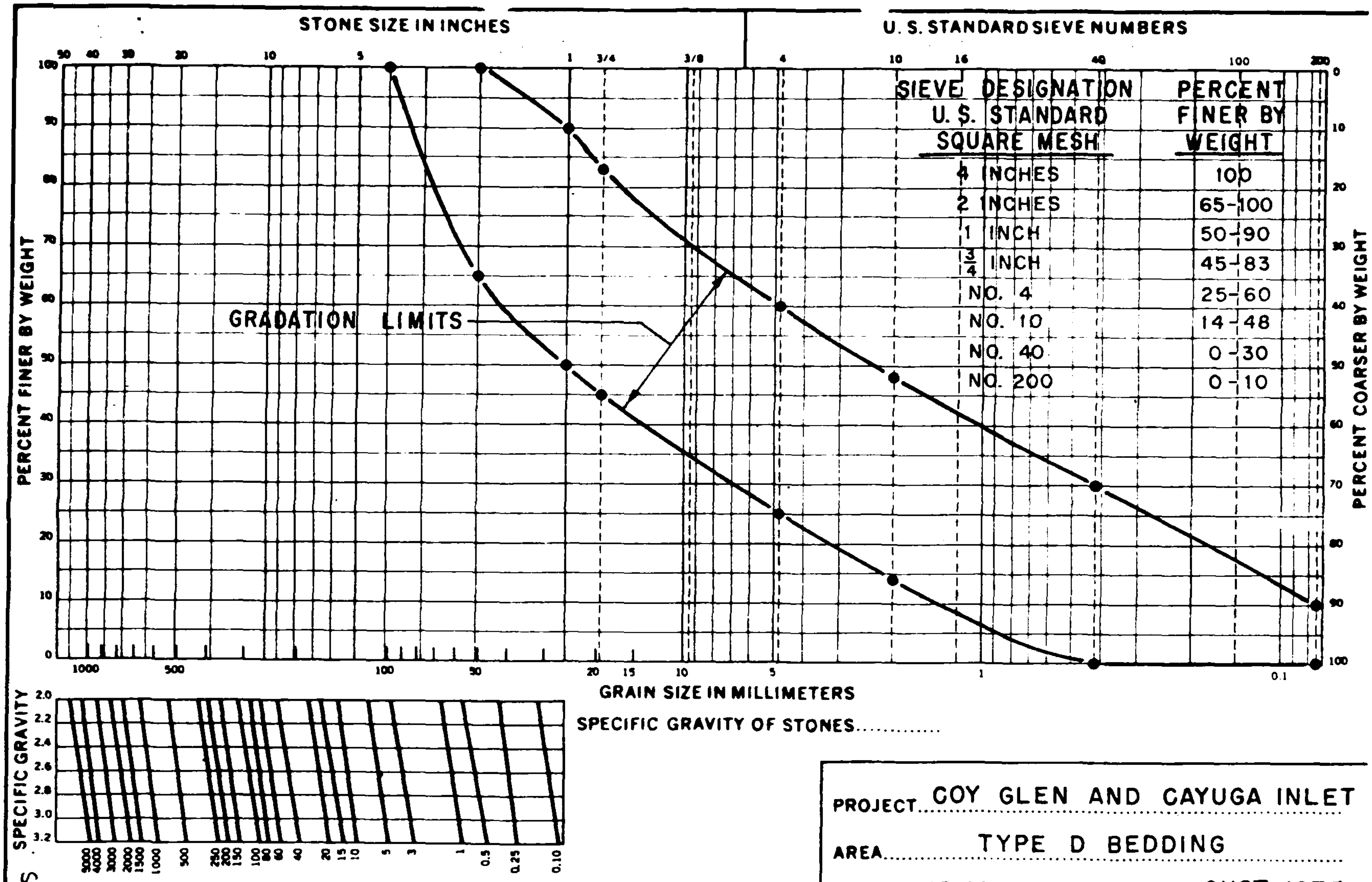
RIPRAP GRADATION CURVES

Sh. R-16

123

ENG FORM 4055
APR 67

PLATE 1



SIEVE DESIGNATION U. S. STANDARD SQUARE MESH	PERCENT FINER BY WEIGHT
4 INCHES	100
2 INCHES	65-100
1 INCH	50-90
$\frac{3}{4}$ INCH	45-83
NO. 4	25-60
NO. 10	14-48
NO. 40	0-30
NO. 200	0-10

SPECIFIC GRAVITY OF STONES............

* ASSUMING STONE SHAPE MIDWAY BETWEEN A SPHERE & CUBE

PROJECT COY GLEN AND CAYUGA INLET

AREA TYPE D BEDDING

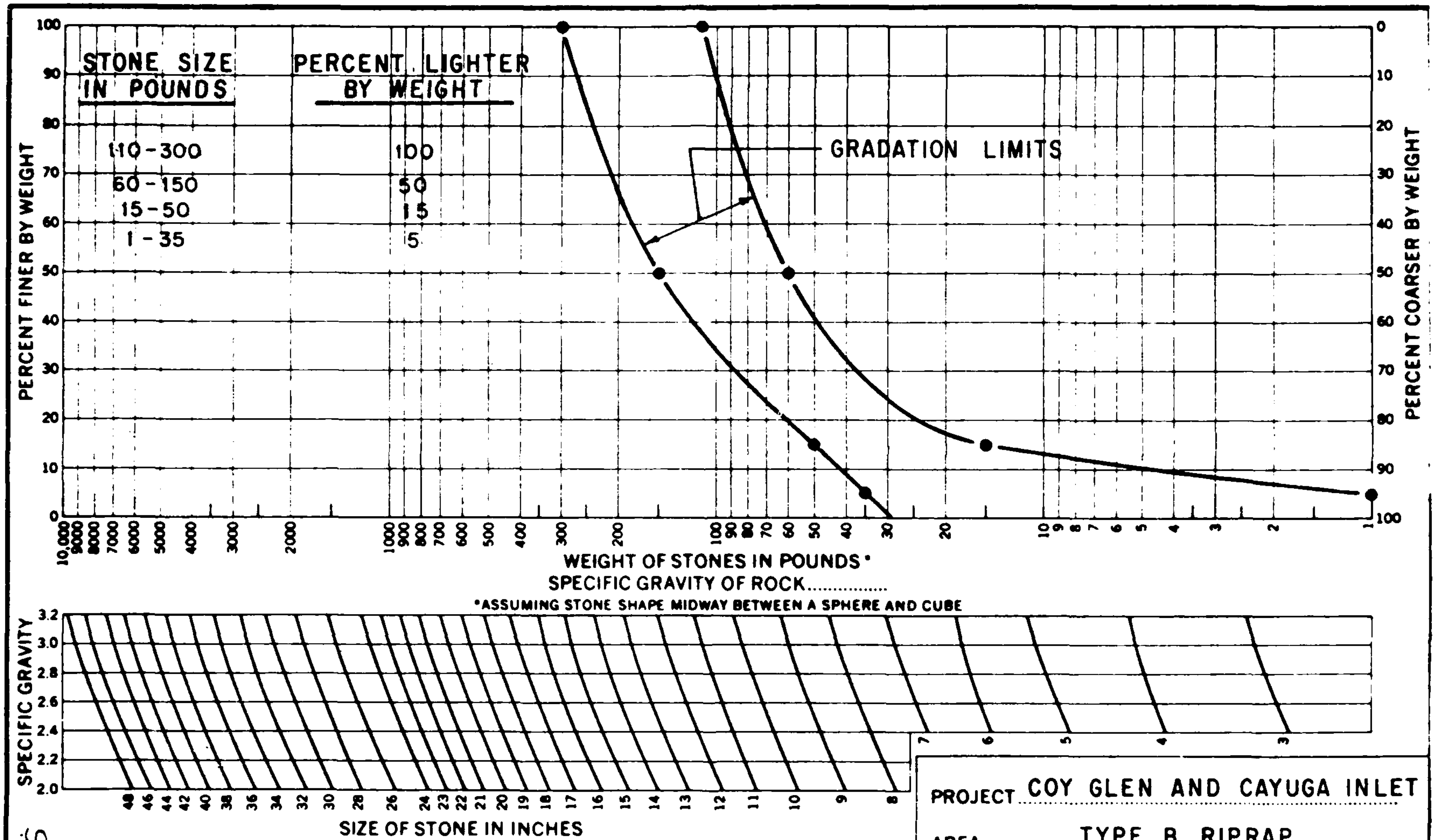
DATE DESIGN ANALYSIS, AUGUST 1975

GRADATION CURVES FOR RIPRAP FILTER AND BEDDING

ENG FORM 4056 APR 67

PLATE 2

Sh R-17



PROJECT COY GLEN AND CAYUGA INLET

AREA TYPE B RIPRAP

DATE DESIGN ANALYSIS, AUGUST 1975

RIPRAP GRADATION CURVES

Sh. R-18

125

ENG FORM 4055 APR 67

PLATE 3

4. HYDRAULIC DESIGN

4.1 Hydraulic calculations were necessary for the design of the drop structure bottom slab and the design of the baffle blocks. Calculations of the hydraulic impact on the bottom slab are on Sheets H-1 and 2. Calculations of the lateral dynamic force on the baffle blocks are on Sheet H-3.

4.2 The Buffalo District Corps of Engineers developed the hydraulic design for Coy Glen which are on pages 132 through 146 and include the following items:

Hydraulic Design Methodology - page 132

Hydraulics of Spillways - page 137

Drop Structures and Check Dams - page 142

Drop Structure Correspondence - page 145

HNTB

MADE BY [illegible] DATE [illegible] JOB NO. [illegible]
CHECKED BY SJK DATE 9/3/75 SEC. NO.
SHEET NO. H-1

CALCULATIONS FOR Nappe Impingement on Base
Dynamic load Development

From Appendix D, Enclosure U-1 to Contract Documents:

Q = Discharge in cfs = 500
D_u = Upstream depth in channel = 4.9' (maximum)
I = Channel Base Width Upstream = 15'
K = Side Slope = 2:1
A = Cross Sectional Area
D = Fall from invert to invert = 10.5' Maximum

The approach velocity of flow over the crest will be increased somewhat due to vertical walls and anticipated flow contraction. If contraction neglected $V = 500/15(4.9) = 6.80$

h_e = Approach velocity head = $\frac{V^2}{2g}$

For path of impingement figure 288-D-2437 page 411, "Design of Small Dams", 2nd Edition 1973 has been used. Notation follows accordingly until otherwise indicated.

H_e = Depth upstream = 4.9'
h_d = Surface upstream to surface downstream = 9.5 max
Y = Crest to base = 10.5' max
g = Acceleration due to gravity = 32.2 ft/sec²
q = Discharge / unit width = 500/15 = 33.33 cfs/ft
V_1 = 33.33/4.9 = 6.80 fps ignoring end contractions

CONSULTING ENGINEERS HOWARD NEEDLES TAMMEN & BERGENDOFF

HNTB

CALCULATIONS FOR

MADE BY SIM DATE 4/2/75 JOB NO. 4204-94-01
CHECKED BY SJK DATE 4/3/75 SEC. NO.
SHEET NO. H-2

Drop Number $\bar{D} = \frac{q^2}{gY^3} = \frac{33.33^2}{32.2(10.5)^3} = .03$

$h_0/H_e = 9.5/49 = 1.94$

Froude Number $F_1 = \frac{V_1}{\sqrt{d_1 g}} = \frac{6.80}{\sqrt{4.9(32.2)}} = 0.54$

From 288-D-2437

$L_P/Y = 1.2^{\pm}$

$L_P = 1.2(10.5) = 12.6'$

Gross assumptions -

1. No nappe contraction ∴ width of nappe equals depth = 4.9'

2. Full impact on base of structure

3. Full velocity buildup due to gravity

~~4. Time of travel = 12.6/6.8 = 1.85 seconds~~

~~5. Vertical velocity = ½ at² = 16.1 (1.85)² = 55.1 fps~~

~~According to "Design of Small Dams" a conservative approximation of dynamic force = 2wA(SE) where SE = specific energy, w = unit weight of water, A = impingement area~~

~~∴ $\frac{Force}{Area} = \frac{2(62.4)(55.1)^2}{64.4} = 5883 \#/ft^2$ Say 6000 psf~~

Deletion afm

128

Page ___ of ___ pages.

Subject Coy Glen Sheet H-2A

Computation of

Computed by AJA Checked by RJG Date 1/12/76

Assume all water falls from the average height of the upstream water depth above the crest down to the top of the base slab.

Average height above crest = $\frac{4.9'}{2}$ = 2.45'

Distance from crest to top of slab = 10.50'

Total average fall 12.95' use 13'

$s = \frac{1}{2}at^2$ where: s = fall distance, a = acceleration due to gravity, t = time

$t = \sqrt{\frac{2s}{a}} = \sqrt{\frac{2 \times 13}{32.2}} = .90$ seconds

$V = at$ where V = velocity

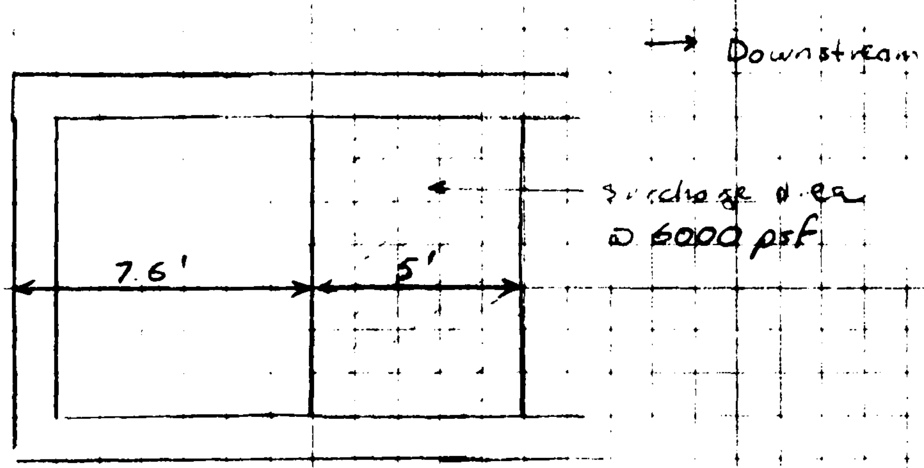
$= 32.2\ '/sec^2 \times .90\ sec = 29.0\ '/sec$

According to "Design of Small Dams" a conservative approximation of dynamic force = 2 w.A.(S.E.) where S.E. = specific energy $\left(\frac{V^2}{2g}\right)$ w = unit weight of water

A = Impingement Area

$\therefore \frac{Force}{Area} = 2(62.4)\frac{(29)^2}{64.4} = 1{,}630\ psf$

Location of dynamic surcharge.



PLAN - NOT TO SCALE

LOAD ON IMPACT BLOCKS

"Design of Small Dams" suggests an impinging force on the upstream face of baffle blocks on page 397 as follows:

Force = 2wA $(d_1 + h_{v_1})$

where: w = unit weight of water
A = upstream face area
$(d_1 + h_{v_1})$ = Specific Energy entering the basin

Considering ... block face H × E (D-1 given) = 2.6 × 1.3
= 3.38'²

2) d_1 = 4.9' ±
3) v_1 = 6.80' ; h_{v_1} = 0.72' ∴ SE = 5.62'

Horiz F on each block = 2 (62.4) 3.38 (5.62) = 2370
Say 3000 #

Subject: COY GLENN, ITHACA N.Y. – 50 YR DESIGN Q = 500 CFS; 100 YR DESIGN Q = 1000 CFS

Computation of: OBJECTIVES & CONCLUSIONS

Computed by: R.S.P. Checked by: ______ Date: 17 JUNE 72

SCHEME – 1 & 1A – 2 & 2A

THE PURPOSE OF THIS HYDRAULIC DESIGN IS TO PROVIDE STABILIZATION TO THE COY GLENN DRAINAGE BASIN. CRITICAL SLOPES HAVE CAUSED EXCESSIVE VELOCITIES AT LOW FLOWS AND CONSEQUENTLY THE CHANNEL HAS BECOME SEVERELY ERODED. TWO SCHEMES ~~OF~~ FOR SOLVING THE PROBLEM WILL BE EXAMINED.

1] USING TWO DROP STRUCTURES TO DISSIPATE ENERGY

2] ONE DROP STRUCTURE TO DISSIPATE ENERGY

RESULTS ARE: SCHEME 1

1] TWO IDENTICAL DROP STRUCTURES @ STA 2+00 AND 3+28 WOULD BE NEEDED. DIMENSIONS ARE:

L_{BASIN} = 24 FT
L TO U.S. FACE OF BLOCKS = 18.2 FT
BLOCK HT. = 2.6 FT
WIDTH & SPACING = 1.3 FT
END SILL HT = 2 FT
Y = 10.5 FT [SEE PG. 2-4]

SEE SHEET 1 FOR BACKWATER

SCHEME 2

2] ONE DROP STRUCTURE AT STA 1+22.5

L_{BASIN} = 27.5 FT
L TO U.S. FACE OF BLOCKS = 21.8 FT
BLOCK HT = 2.6 FT
WIDTH & SPACING = 1.3 FT
END SILL HT = 2 FT
Y = 19.1 FT

SEE SHEET 1 FOR BACKWATER

SCHEME – 1-A

1] TWO D.S. AS IN SCHEME 1 ONLY THE CHANNEL BOTTOM IS ONLY 15' WIDE – 50 YEAR DESIGN Q = 500 CFS

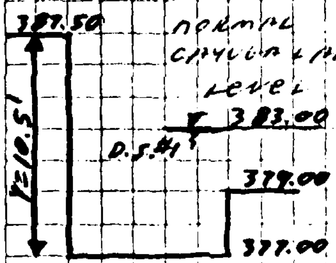
SCHEME – 2-A

1] ONE D.S. AS IN SCHEME 2 ONLY THE CHANNEL BOTTOM IS ONLY 15' WIDE – 50 YEAR DESIGN Q = 500 CFS

GPO 946-479

Subject: COY GLEN, ITHACA N.Y. - 100 YR DESIGN Q = 1000 CFS
Computation of: BASIN DIMENSIONS - CONCRETE D.S. AT STA 0+00
Computed by: B.S.A. Checked by: ______ Date: 17 JUNE 72

SCHEME - 1 [redacted]
RECTANGULAR WEIR - 30 FEET WIDE
CREST ELEVATION = 387.50



$d_c = 3.27$ AREA $= 3.27 \times 30 = 98.1$ □'
$V_c = 1000/98.1 = 10.19$ FT/SEC
$V_c^2/2g = 1.61$ $H_e = 1.61 + 3.27 = 4.88$

REQUIRED T.W. $= 1.8 d_c \approx 6.00'$

a] ENERGY LINE ELEVATION AT CREST OF DROP = CREST ELEV $+ d_c + \frac{V^2}{2g}$
$= 387.50 + 3.27 + 1.61 = 392.38$

b] T.W. ELEVATION = 383.00 & $h_d = 392.38 - 383.00 = 9.38$

c] $\frac{h_d}{H_e} = \frac{9.38}{4.88} = 1.92$ d] DROP # $= D = \frac{q^2}{gY^3}$

e] $q = 1000/30$ $D = \frac{[1000/30]^2}{[32.2][10.5]^3} = 0.03$

f] $Y = 387.50 - 377.00 = 10.5$ FT

FROM CURVE PG 304 OF "DESIGN OF SMALL DAMS"

FOR $D = 0.03$ AND $\frac{h_d}{H_e} = 1.92$

$L_p/Y = 1.3 \rightarrow L_p = 10.5 \times 1.3 = 13.65$

$L_{BASIN} = L_p + 2.55 d_c = 13.65 + 8.34 = \underline{22.0 \text{ FT}}$

L TO U.S. FACE OF BLOCKS $= L_p + .8 d_c = 13.65 + 2.62 = \underline{16.20 \text{ FT}}$

BLOCK HT $= 0.8 d_c = 2.6$ FT

WIDTH & SPACING $= 0.4 d_c = 1.3$ FT

GO TO PAGE 3

NOTE - THESE CALCULATIONS ARE ALSO FOR SCHEME 1-A
Q = 500 CFS

GPO 946-479

Subject COY GLEN, ITHACA, N.Y. - 100 YR DESIGN Q = 1000 CFS

Computation of BASIN DIMENSION - AFTER EM 1110-345-284

Computed by B.S.P. Checked by Date 17 June 72

SCHEME - 1 ; 1A

$h = 8.5 \quad d_c = 3.27$

$h/d_c = 8.5/3.27 = 2.60 \Rightarrow$ FROM FIG 24 - Pg 64 $\frac{h'}{d_c} = 0.50$

SO $h' = 0.5 \times 3.27 = 1.63$

FOR $\frac{h}{d_c} = 2.60 \Rightarrow C_L = 3.8 = \frac{L}{\sqrt{h d_c}}$

$L = 3.8 \times \sqrt{8.5 \times 3.27} = 3.8 \times \sqrt{27.79} = 3.8 \times 5.27 = 20.02$

REDUCE 20.02 BY 10% = 20.02/1.1 = 18.2 FT

FROM DESIGN OF S.D. - LENGTH TO BAFFLES = 16.2 FT
" EM 1110-345-284 - " " " = 18.2 FT

SO BASIN SHOULD BE 2.0 FT LONGER THAN THAT DETERMINED FROM DESIGN OF S.D.

$L_B = 22 + 2 = 24$ FT

NOTE - USE SAME SIZE DROP STRUCTURE AT STA 1+24.
THAT IS;

L BASIN = 24 FT.

L TO U.S. FACE OF BLOCKS = 18.2 FT.

BLOCK HT = 2.6 FT.

WIDTH & SPACING = 1.3 FT

END SILL HT = 2 FT

GPO 946-479

		FLOW LINE COMPUTATIONS UNDER IMPROVED CONDITIONS	STREAM: Coy Glen, Ithaca, N.Y.	SHEET NO. 1 of 2
SLOPE = 0.005	STA 0+00 to 2+00		DISCHARGE Q = 500 Cu. Ft. Per. Sec.	DATE 7/10/75

CHANNEL BOTTOM ELEVATION	DEPTH OF WATER	MILE OR STATION	LENGTH OF REACH L	TRIAL ELEVATION	AREA A	BOTTOM WIDTH	HYDR RADIUS R	COEF. OF ROUGH n	$S^{1/2}$ = .01 VELOCITY v	$S^{1/2}$ = .01 DISCHARGE q	SLOPE S	MEAN SLOPE Sm	VELOCITY V	DISCHARGE Q	V^2Q	FRICTION HEAD h_f	VELOCITY HEAD h_v	$h_{v_1}-h_{v_2}$	TOTAL HEAD H	WATER SURFACE ELEVATION	ENERGY GRADIENT
	dc		Start	at	d/c	STATION	0	+00	For	[illegible]	Design										
378.00	2.85	0+00	–	380.85	59	15 V/1:2	2.13	0.045	0.55	32.45	0.023741		8.47	500		–	1.16	–	–	380.85	382.01
												0.016442						+.64+.22			
378.05	3.70	0+10	10	381.75	83	15 V/1:2	2.63	0.045	0.63	52.29	0.009143		6.02	500		0.16	0.56	0.72	0.88	381.73	382.29
												0.008697									
378.10	3.80	0+20	10	381.90	86	15 V/1:2	2.68	0.045	0.64	55.04	0.008252		5.81	500		0.09	0.53	+0.03	+0.12	381.85	382.38
												0.007861									
378.20	3.90	0+40	20	382.10	89	15 V/1:2	2.74	0.045	0.65	57.85	0.007470		5.62	500		0.16	0.49	+0.04	+0.20	382.05	382.54
												0.006634						+.004.22			
379.00	4.20	2+00	160	383.20	98	15 V/1:2	2.91	0.045	0.67	65.66	0.005798		5.10	500		1.06	0.40	+0.11	1.17	383.22	383.62
			START	AT	383.00	STATION			0+00	FOR	DESIGN	PROFILE									
378.00	5.00	0+00	–	383.00	125	15 V/1:2	3.35	0.045	0.74	92.5	0.00292		4.00	500		–	0.25	–	–	383.00	383.25
												0.003101						-.03+.4			
378.50	4.80	1+00	100	383.30	118	15 V/1:2	3.24	0.045	0.72	84.96	0.00346		4.24	500		0.32	0.28	-.02	+0.30	383.30	383.58
												0.003555									
379.00	4.70	2+00	100	383.70	115	15 V/1:2	3.18	0.045	0.72	82.8	0.003646		4.35	500		0.36	0.29	-0.01	+0.35	383.65	383.94

NOTES: $R = A/P$ $v = \frac{1.486}{n} R^{2/3} S^{1/2}$ where $S^{1/2} = .01$ $S = \left(\frac{Q \times .01}{\text{Sum } q}\right)^2$ $Q = AV$ $h_f = S_m L$

COMPUTED BY B.S.R.

CHECKED BY

scheme 1 - fill above #1 Drop Structure to eleve 397.00

Slope STA 2+28 to 3+28 = 0.01
Slope STA 3+56 to 6+00 = 0.00
Slope STA 6+00 → = Natural Conditions

FLOW LINE COMPUTATIONS UNDER IMPROVED CONDITIONS

STREAM: Coy Glen

DISCHARGE Q = 500 Cu. Ft. Per. Sec.

SHEET NO. 2 of 2

DATE: July 1, 1975

Channel Bottom Elevation	Depth of Water	Mile or Station	Length of Reach L	Trial Elevation	Area A	Bottom Width	Hydo Radius R	Coef. of Rough n	$s^{\frac{1}{2}} = .01$ Velocity v	Discharge q	Slope S	Mean Slope Sm	Velocity V	Discharge Q	V^2Q	Friction Head h_f	Velocity Head h_v	$h_{v_1}-h_{v_2}$	Total Head H	Water Surface Elevation	Energy Gradient
		START		at Critical		Depth	at		the	Crest	of the	Downstream		Drop		Structure		#	2		
												—									
397.5	3.27	2+28	—	390.77	49.05	15 ⊔ Rect	2.28	.035	.74	36.30	.01897		10.19	500			1.61			390.77	392.38
			5									.0105				.05		1.33+.2x			
397.55	4.30	2+33		392.35	118	15 ⊔ 1:2	3.24	.035	.93	110	.00288		4.24	500			.28	1.65	1.65	392.42	392.70
			10									.00208				.02					
387.16	4.30	2+43		392.45	118	15 ⊔ 1:2	3.24	.035	.93	110	.00208		4.24	500			.29		.02	392.44	392.72
			55									.00295				.25		-.15			
388.50	4.10	3+28		392.65	95.1	15 ⊔ 1:2	2.95	.035	.85	80.8	.00383		5.36	500			.43		.10	392.54	392.97
			START	at Critical		Depth	at		the	Crest	of the	upstream		Drop		Structure		#1			
			—																		
397.00	3.27	3+56		400.27	49.05	15 ⊔ Rect.	2.28	.035	.74	36.30	.01897		10.19	500			1.61			400.27	401.88
			10			x EXIST						.00962				.10		1.56+.2x			
397.00	5.30	3+66		402.30	268	40 ⊔ 1:2	4.21	.035	1.11	297	.000253		1.87	500			.05	1.87	1.97	402.24	
			34									.000283				.01					
397.00	5.30	4+00		402.30	268	40 ⊔ 1:2	4.21	.035	1.11	297	.000283		1.87	500			.05		.01	402.25	
			100									[illegible]				[illegible]					
397.00	5.30	5+00		[illegible]	268	40 ⊔ 1:2	[illegible]	.035	1.11	297	[illegible]		1.87	500			.05		[illegible]	402.28	
			100									[illegible]				.02					
397.00	5.30	6+00		402.30	268	40 ⊔ 1:2	[illegible]	.035	1.11	297	[illegible]		1.87	500			[illegible]		[illegible]	402.31	
			—																		
397.00	5.30	[illegible]		402.30	268	[illegible]	4.21	.045	.96	230	[illegible]		1.87	500			.05			402.31	
			100									.005815				.58		-.44+.4x			
400.50	2.10	7+00		402.60	92.9	40 ⊔ 1:2	1.88	.045	.51	47.3	.011610		5.39	500			.45	-.24	+.34	402.65	
			100									.021460				2.15		-.42+.4x			
403.00	1.55	8+00		404.55	66.80	40 ⊔ 1:2	1.43	.045	.42	20.0	.031725		7.48	500			.87	-.25	+1.90	404.55	
			100									.020735				2.07		[illegible]			
405.00	2.20	9+00		407.20	97.10	40 ⊔ 1:1	1.96	.045	.52	50.0	.00969		5.12	500			.41	-.55	2.62	407.17	

NOTES: $R = A/P$ $v = \frac{1.486}{n} R^{2/3} s^{\frac{1}{2}}$ where $s^{\frac{1}{2}} = .01$ $S = \left(\frac{Q \times .01}{\text{Sum } q}\right)^2$ $Q = Av$ $h_f = S_m L$

COMPUTED BY: CMM

CHECKED BY: B.S.P.

structural design. The structure must be made sufficiently stable to resist sliding against the impact load on the baffle wall. The entire structure must resist the severe vibrations inherent in this type of device, and the individual structural members must be sufficiently strong to withstand the large dynamic loads.

Riprapping should be provided along the bottom and sides adjacent to the structure to avoid the tendency for scour of the outlet channel downstream from the end sill when a shallow tailwater exists. Downstream wingwalls placed at 45° may also be effective in reducing scouring tendencies and flow concentrations downstream.

Figure 218. **An impact type stilling basin in operation.**

203. *Plunge Pools.*—When a free-falling overflow nappe drops vertically into a pool in a riverbed, a plunge pool will be scoured to a depth which is related to the height of the fall, the depth of tailwater, and the concentration of the flow [13]. Depths of scour are influenced initially by the erodibility of the stream material or the bedrock and by the size or the gradation of sizes of any armoring material in the pool. However, the armoring or protective surfaces of the pool will be progressively reduced by the abrading action of the churning material to a size which will be scoured out and the ultimate scour depth will, for all practical considerations, stabilize at a limiting depth irrespective of the material size. An empirical approximation based on experimental data has been developed by Veronese [14] for limiting scour depths, as follows:

$$d_s = 1.32\, H_T^{0.225}\, q^{0.54} \qquad (26)$$

where,

d_s = the maximum depth of scour below tailwater level in feet,

H_T = the head from the reservoir to tailwater levels in feet, and

q = the discharge in second-feet per foot of width.

F. HYDRAULICS OF SPILLWAYS

204. *Free Overfall (Straight Drop) Spillways.*—(a) *General.* The hydraulic problems of the free overfall spillway are concerned with the characteristics of the control and with the dissipation of flow in the downstream basin. Flow over the control ordinarily is free discharging; air is admitted to the underside of the nappe to avoid the jet being depressed by reduced underneath pressure. Dissipation of the flow in the downstream basin may be obtained by the hydraulic jump, by impact and turbulence induced in a basin with impact blocks, or by a slotted grating dissipator installed immediately downstream from the control.

The control may be either sharp crested to provide a fully contracted vertical jet, broad crested to effect a fully suppressed jet, or shaped to increase the crest efficiency. Coefficients of discharge will approximate those indicated in section 190. The sides of the control usually are arranged to allow for full side contraction, in order to provide side space for the access of air to the underside of the nappe. This contraction is effected by providing square abutment headwalls or by installing square-cornered vertical offsets along the piers or walls opposite the crest. The effective length of the crest is then determined according to

135

equation (4) where K_p and K_a will approximate 0.20.

The dimensions of the stilling basin for the free overfall spillway can be related to two independent variables; namely, the drop distance Y and the unit discharge q. These variables, which are dimensional terms, can be expressed in a dimensionless ratio by expressing q in lineal form by means of the equation for critical depth, $d_c=\sqrt[3]{\frac{q^2}{g}}$, as follows:

$$\frac{d_c}{Y}=\sqrt[3]{\frac{q^2}{gY^3}}$$

From this expression it can be seen that $\frac{q^2}{gY^3}$ is a dimensionless ratio which can be used as an independent variable to which the individual dimensions may be related. This ratio is called the "drop number" and is designated $\overline{D}$. It can be shown that $\overline{D}$ is the product of F_1^2 and $\left(\frac{d_1}{Y}\right)^3$, where F_1 is the Froude number $\frac{v_1}{\sqrt{d_1 g}}$ at the point where the nappe meets the basin floor.

(b) *Hydraulic Jump Basin.*—The jump characteristics of the straight drop basin are basically the same as those for other jump basins, except that the position of the start of the jump cannot be determined as readily as it can for other basins. On figure 219 the point of the start of the jump (point X) will vary with the vertical drop distance and is influenced by the under nappe pool depth, d_f. The basin design downstream from point X will be patterned after those discussed in section 199, once distance L_d is determined. Values of the depth d_1, and of the Froude number, F_1, at the start of the jump in relation to the drop number, $\overline{D}$, are shown on figure 219. These relations may be used for determining the basin dimensions.

Where tailwater depths are greater than the conjugate depth d_2, the jump will move back on the free falling nappe raising the depth d_f of the under nappe pool. With greater depths of the under nappe pool, the nappe will not plunge immediately to the floor of the basin but will be deflected upward along the top of the under pool so that it will meet the floor to the right of point X. The distance to the start of the jump, L_d, will become progressively longer as the tailwater depth is increased. Average values of L_d in relation to $\frac{h_d}{H_e}$, as determined from tests, are plotted on figure 219. For a basin with excessive depths the type II basin discussed in section 199 is most adaptable. The impact block type basin, discussed below, also can be adopted for low drop spillways with excessive tailwater depths.

(c) *Impact Block Type Basin.* An impact block basin has been developed [1] for low heads which gives reasonably good dissipation of energy for a wide range of tailwater depths. The dissipation of the high energy is principally by turbulence induced by the impingement of the incoming flow upon the impact blocks. The required tailwater depths, therefore, become more or less independent of the drop height. The linear proportions are as follows:

Minimum basin length, $L_B=L_p+2.55\ d_c$

Minimum length to upstream face of baffle block $=L_p+0.8\ d_c$

Minimum tailwater depth, $d_{tw}=2.15\ d_c$

Optimum baffle block height $=0.8\ d_c$

Width and spacing of baffle block $=0.4\ d_c\pm$

Optimum height of end sill $=0.4\ d_c$

(d) *Slotted Grating Dissipator.*—An effective dissipator for small drops is illustrated on figure 220. This device has been tested for values of the Froude number, F_1, as determined at basin apron level, in the range of 2.5 to 4.5. For this arrangement the overfalling sheet is separated into a number of long, thin segments which fall nearly vertically into the basin below, where dissipation of energy takes place by turbulence. To be effective the length of the grating, L_G, must be such that the entire incoming flow will fall through the slots before reaching the downstream end. The length is therefore a function of the total discharge, the velocity of the incoming flow, and the area of the grating slots. Experimental tests indicate that the following relation gives an effective design:

$$L_G=\frac{Q}{0.245wN\sqrt{2gH_e}} \qquad (27)$$

where:

L_G = the length of the grating in feet,
w = the width of the slot in feet,
N = the number of slots, and
H_e = the depth of flow upstream from the drop.

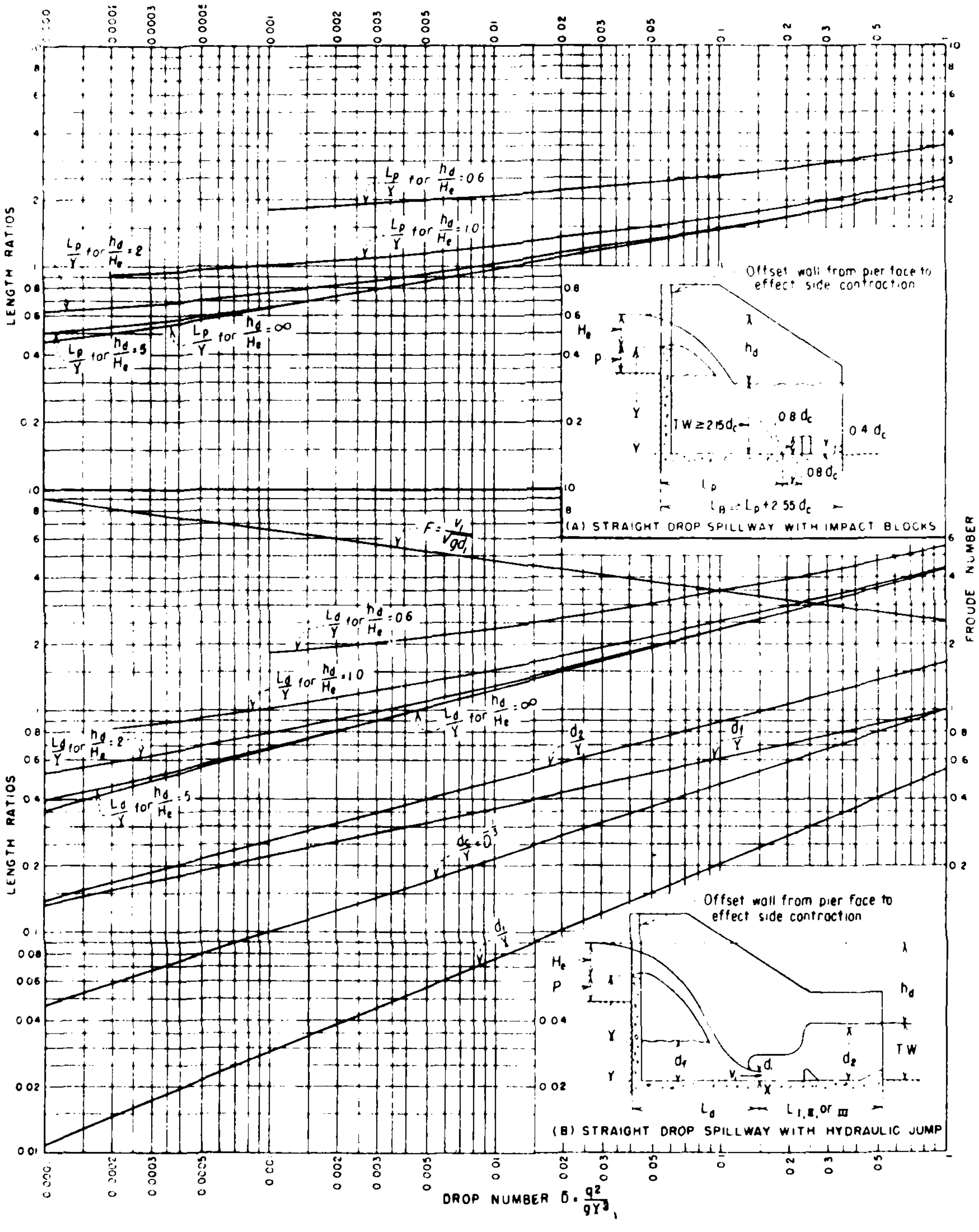


Figure 219. **Hydraulic characteristics of straight drop spillways with hydraulic jump or with impact blocks.**

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The length of the basin, L_B, should be approximately 1.2 L_G. An end sill similar to that for basin type I, discussed in section 199, can be provided to improve the hydraulic action.

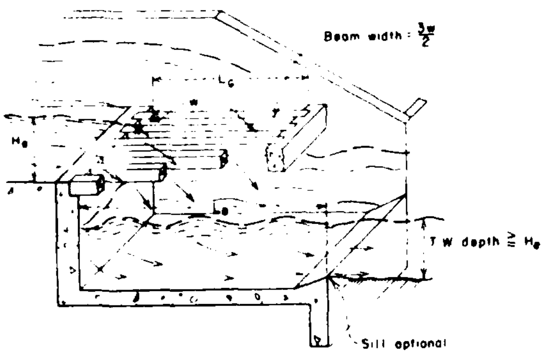


***Figure 220.* Slotted grating dissipator.**

(e) *Example of Design of a Free Overfall Spillway.* The procedure for designing a free overfall spillway is best shown by means of an example. Consider that such a spillway is to be designed to discharge 500 second-feet. The drop from the spillway crest to the tailwater level for a flow of 500 second-feet is 12 feet. (Tailwater elevation is 108.0.) The approach channel is 20 feet long and the approach floor is level with the spillway crest which is at elevation 120.0. Each type of energy dissipator is to be investigated.

The procedure for design of the *hydraulic jump basin* is as follows: First, assume the effective length of the spillway crest to be 15 feet. Assume an approximate value of $C=3.0$. The unit discharge, q, is equal to $\frac{500}{15}=33.3$ second-feet and H_e is equal to $\left(\frac{q}{C}\right)^{2/3}=\left(\frac{33.3}{3.0}\right)^{2/3}=5.0$ feet. The reservoir water surface elevation, therefore, is 120.0 +5.0=125.0. Thus the drop from reservoir level to tailwater level will be approximately 17 feet.

Assume that an offset of 0.5 foot is provided along each side of the weir to effect side contractions for aerating the underside of the sheet, and that the offset is square-cornered. Then the net crest length, which will also be the stilling basin width, is:

$$L'=L+2K_aH_e+2(0.5)=15+2(0.2)(5)+1.0=18.0 \text{ feet.}$$

Figure 208 is used to determine the approximate apron level of the jump basin, assuming the effective width of the basin to be 15 feet and (for the first trial) that there will be no loss of energy between the reservoir and the point where the jet strikes the basin floor. From scale A, the conjugate depth d_2 for $q=33.3$ second-feet and $H_T=17$ feet is 8.8 feet, which places the apron floor at elevation 99.2. Y is equal to elevation 120 − elevation 99.2=20.8 feet, and the drop number $\overline{D}$ is equal to $\frac{q^2}{gY^3}=\frac{33.3^2}{32.2\times 20.8^3}=0.0038$. For $\overline{D}=0.0038$, from figure 219 $\frac{d_2}{Y}=0.375$ and $d_2=7.8$ feet. The apron level then must be adjusted to an elevation which is d_2 below the tailwater elevation 108.0, or elevation 100.2.

For the second trial, the adjusted value of Y is 19.8 and $\overline{D}$ is equal to $\frac{33.3^2}{32.2\times 19.8^3}=0.0044$. For $\overline{D}=0.0044$ and $\frac{h_d}{H_e}=\frac{17}{5}=3.4$, from figure 219, $\frac{L_d}{Y}=1.02$ and $L_d=20.2$ feet. Also $d_1=1.1$ feet and $F_1=5.3$.

With the values of $F_1=5.3$, $d_1=1.1$ and $d_2=7.8$, the arrangement of the type II basin shown on figure 206 can be used. From figure 206, $\frac{L_{II}}{d_2}=2.37$ and $L_{II}=18.5$ feet. The length of the basin measured from the vertical crest is equal to $L_d+L_{II}=20.2+18.5=38.7$ feet. The distance of the baffle blocks from the vertical crest for this basin will be 20.2 feet plus 0.8 d_2 or 20.2 plus 0.8 (7.8)=26.4 feet, approximately.

The baffle blocks will be approximately 1.5 d_1 or 1.6 feet high and will be about 14 inches wide and spaced at about 28-inch centers.

For the *impact block basin*, the procedure is as follows: The critical depth, d_c, is equal to $\sqrt[3]{\frac{q^2}{g}}=\sqrt[3]{\frac{33.3^2}{32.2}}=3.3$ feet. Then from figure 219, for $\overline{D}=0.0044$ and $\frac{h_d}{H_e}=3.4$, $\frac{L_p}{Y}=0.85$ and $L_p=17.0$ feet. The minimum length of the basin, L_B, is equal to $L_p+2.55\,d_c=17.0+2.55\,(3.3)=25.4$ feet, say 26 feet. The minimum tailwater depth of 2.15 d_c will be 7.1 feet which places the basin

floor at elevation 100.9. The distance from the vertical crest to the baffle blocks will be $L+0.8\ d_c=17.0+0.8\times3.3=19.6$ feet, say 20 feet. The baffle blocks will be about $0.8\ d_c$ or 3.0 feet high and about 18 inches wide, spaced at about 3-foot centers. The end sill will be $0.4\ d_c$ or about 1.5 feet high.

It can be seen from the above result that if the impact block basin is used, the basin can be made almost 13 feet shorter than that required for a hydraulic jump basin, and also that the impact block basin will be 0.7 foot shallower. The baffle blocks for the hydraulic jump basin will be smaller and spaced closer together than those for the impact block basin.

This example shows that the impact block basin is considerably smaller than the hydraulic jump basin. However, the impact block basin should be limited to use where the drop distance does not exceed 20 feet. Furthermore, as previously explained, the foundation for an impact block basin must be of better quality because of the concentrated forces involved. The hydraulic jump basin, therefore, has a much wider application of use.

The *slotted grating dissipator* is not suitable in this case because the Froude number of 5.3 is in excess of the 4.5 value, which is the tested limit for a practical slotted grating design.

205. *Drop Inlet (Shaft or Morning Glory) Spillways.*—(a) *General Characteristics.*—Typical flow conditions and discharge characteristics of a drop inlet spillway are represented on figure 221. As illustrated on the discharge curve, crest control (condition 1) will prevail for heads between the ordinates of a and g; orifice or tube control (condition 2) will govern for heads between the ordinates of g and h; and the spillway conduit will flow full for heads above the ordinate of h (represented as condition 3).

Flow characteristics of a drop inlet spillway will vary according to the proportional sizes of the different elements. Changing the diameter of the crest will change the curve ab on figure 221 so that the ordinate of g on curve cd will be either higher or lower. For a larger diameter crest, greater outflows can be discharged over the weir at low heads and the transition will fill up and tube control will occur with a lesser head on the crest. Similarly, by altering the size of the throat of the tube, the position of curve cd will change, indicating the heads above which tube control will prevail. If the transition is made of such size that curve cd is moved to coincide with or lie to the right of point j, the control will shift directly from the crest to the downstream end of the conduit. The details of the hydraulic flow characteristics are discussed in following subsections.

(b) *Crest Discharge.*—For small heads, flow over the drop inlet spillway is governed by the characteristics of crest discharge. The vertical transition beyond the crest will flow partly full and the flow will cling to the sides of the shaft. As the discharge over the crest increases, the overflowing annular nappe will become thicker, and eventually the nappe flow will converge into a solid vertical jet. The point where the annular nappe joins the solid jet is called the crotch. After the solid jet forms, a "boil" will occupy the region above the crotch; both the crotch and the top of the boil become progressively higher with larger discharges. For high heads the crotch and boil may almost flood out, showing only a slight depression and eddy at the surface.

Until such time as the nappe converges to form a solid jet, free-discharging weir flow prevails. After the crotch and boil form, submergence begins to affect the weir flow and ultimately the crest will drown out. Flow is then governed either by the nature of the contracted jet which is formed by the overflow entrance, or by the shape and size of the vertical transition if it does not conform to the jet shape. Vortex action must be minimized to maintain converging flow into the drop inlet. Guide piers are often employed along the crest for this purpose [5, 6, 22].

If the crest profile and transition conform to the shape of the lower nappe of a jet flowing over a sharp-crested circular weir, the discharge characteristics for flow over the crest and through the transition can be expressed as:

$$Q=CLH^{3/2} \qquad (3)$$

where H is the head measured either to the apex of the under nappe of the overflow, to the spring point of the circular sharp-crested weir, or to some other established point on the overflow. Similarly,

energy dissipators apply also to storm-drain outfalls. Generally, however, the range of exit velocities is likely to be more limited for storm drains, and elaborate structures for energy dissipation are rarely required. If the storm drain discharges into a large stream channel or a lake or ocean where strong hydraulic forces are present, artificial dissipation of effluent energy is rarely required, but particular care must be taken to insure that the outlet structure (1) does not adversely affect the streambank or shore stability, and (2) is not caused to fail as a result of the exterior forces.

c. Channel Outlets. Improved channels, especially the paved ones, commonly carry water at velocities higher than those prevailing in the natural channels into which they discharge. Usually a judicious use of riprap will suffice for dissipation of excess energy. The terminus of a paved channel will require a cutoff wall to preclude undermining. In extreme cases a structure such as a flared transition, stilling basin, or impact device may be required.

2-14. DROP STRUCTURES AND CHECK DAMS. a. Description and Purpose. Drop structures and check dams are designed to check channel erosion by controlling the effective gradient, and to provide for abrupt changes in channel gradient by means of a vertical drop. They also provide a satisfactory means for discharging accumulated surface runoff over fills with heights not exceeding about 5 feet and over embankments higher than 5 feet provided the end sill of the drop structure extends beyond the toe of the embankment. The check dam is a modification of the drop structure used for erosion control in small channels where a less elaborate structure is permissible. The hydraulic design of these structures may be divided into two general phases, design of the notch or weir and design of the overpour basin. It must be emphasized that for a drop structure or check dam to be successful, not only must the structure be designed soundly, but also the structure or series of structures must be so placed as to cause the ditches or channels to become stable. The question of what is a stable grade for the channel must be answered before the height and spacing of the various drop structures can be determined. Also the structure must be designed to preclude flanking.

b. Design Rules. Pertinent features of a typical drop structure are shown in figure 24. (Features of an alternate drop structure are given in paragraph DROP STRUCTURES AND CHECK DAMS of EM 1110-345-283.)

(1) Notation used in the design of drop structures is as follows:

C = weir coefficient = 3.0

C_L = coefficient of basin length = $\frac{L}{\sqrt{hd_c}}$

d_c = critical depth over crest, feet

H = head on weir = $3/2\ d_c$, feet

h = height of drop, feet

h' = height of end sill, feet

L = length of basin, feet

Q = discharge, cubic feet per second

W = length of weir or width of crest, feet

(2) Weir. Discharge over the weir should be computed from the equation $Q = CWH^{3/2}$, using a C value of 3.0. The length of the weir should be such as to obtain maximum use of the available channel cross section upstream from the structure. A trial-and-error procedure should be used to balance the weir height and width with the channel cross section.

(3) Stilling basin length and end sill height should be determined from the design curves in figure 24.

(4) Riprap probably will be required on the side slopes and below the end sill immediately downstream from the structure.

2-15. MISCELLANEOUS STRUCTURES. a. Chutes. The chute provides a satisfactory method of discharging accumulated surface runoff over fills and embankments. A typical design is presented in figure 25, and design charts for chutes constructed of concrete for various gradients and discharges are shown in figure 26. On the basis of laboratory

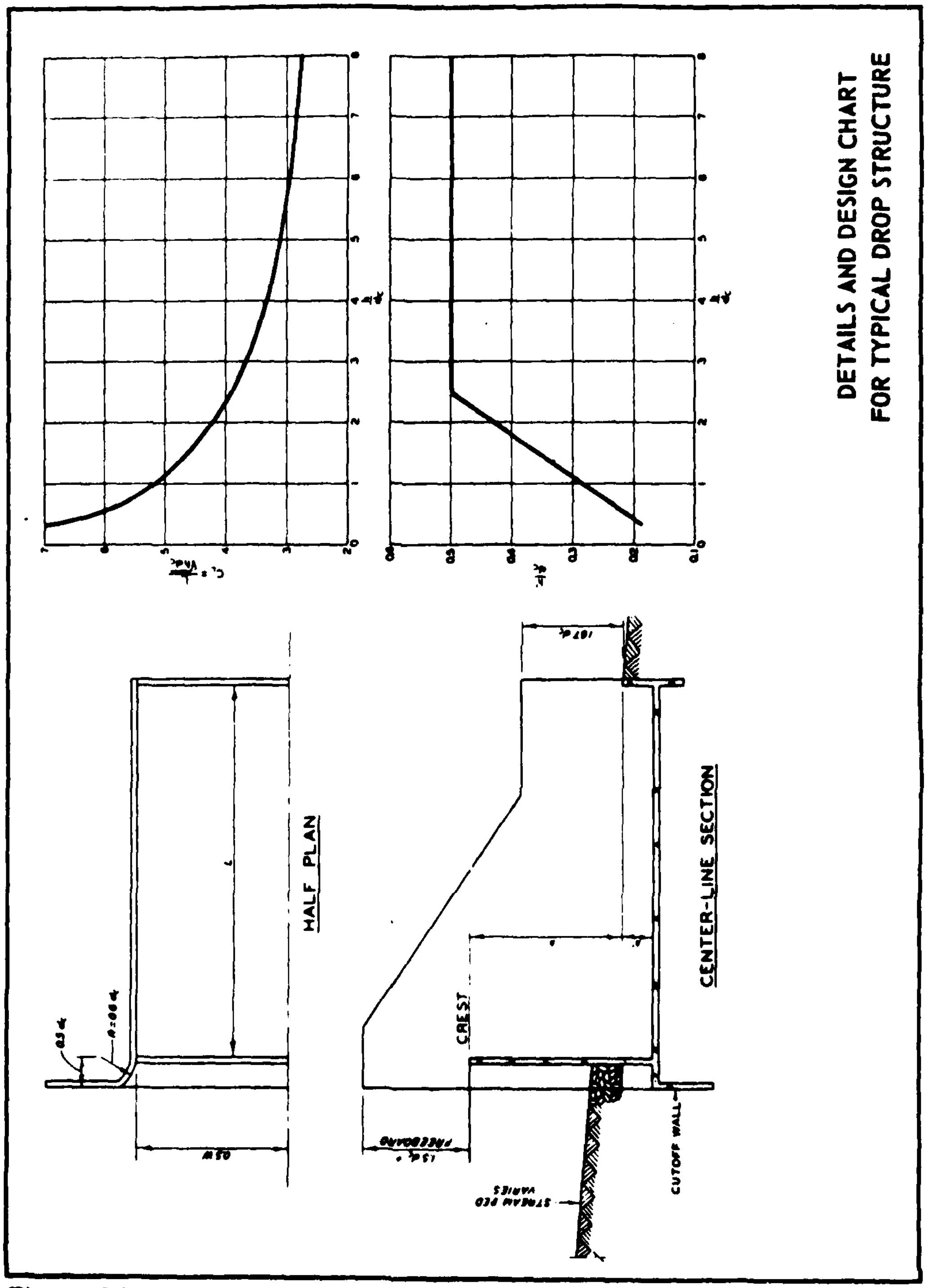


Figure 24

142

Coy Glen - Drop Structure

$Q = 1000$ cfs

$W = 30$ ft

$q = 33.3$ cfs

$d_c = 3.25$

$h = 393.5 - 386.0 = 7.5$ ft

$h/d_c = 7.5/3.25 = 2.3$

From fig. 24, EM 1110-345-284, - TM 5-820-4

$\bar{C}_L = 4.0$

$L = 4.0\sqrt{hd_c} = 19.7$

Since design curve results in a basin length 10 percent greater than minimum acceptable, reduce

$$19.7/1.1 = 17.9 \text{ ft}$$

Thus, I would place the row of baffles 18 ft from the drop rather than 15.6 ft as shown on sketch with letter to Weinrub dated 28 April 1972. If a solid sill is used in place of baffles, its height should be $0.5\ d_c = 1.7$ ft. With baffles, 2.6 ft as shown probably is good.

Also, the above indicate that the basin floor should be at about elev. 384. I would raise the basin to this elevation, use a 2-ft-high end sill, and eliminate the reverse slope on the channel bottom immediately downstream of the end sill. This 1 on 3 reverse slope would require large riprap. If a reverse slope were required, it should be no steeper than 1 on 10.

Further, I would put some rounding on the abutments (see plate 2 of the Gering report, TR 2-760). Also I would terminate the side walls at the end sill. The flared wing walls do more harm than good; use only if required as retaining walls.

Summarizing, I would end up with a basin 24.5 ft long rather than 22 ft, and at elev. 384.0 rather than 382.81. I would place the 2.6-ft-high baffles 18 ft from the drop rather than 15.6 ft. I would use a 2-ft-high end sill. I would round the abutment walls and would eliminate the flared wing walls.

T. E. MURPHY
Chief, Structures Branch
15 May 1972

COY GLEN AND CAYUGA INLET
ITHACA, NY

MATERIAL SURVEY
FOR
DESIGN ANALYSIS

GENERAL

1. A materials survey to determine construction material sources for energy dissipator facilities and riprap repair was performed. Interested sources were investigated.

2. The survey consisted of a preliminary file search in which the following were considered:

a. An analysis of the results of quarry investigations.

b. Laboratory testing of samples and an analysis of the test results, and

c. The evaluation of available service records.

3. The survey included a sufficient number of sources capable of producing the required materials.

MATERIAL DESIGN CRTERIA

4. Material Types and Gradations

a. General. The stone materials for the proposed construction consists of two sizes of riprap, spalls, and bedding. In all cases, no stone shall exceed an elongation ratio of 3:1.

b. Type A Stone. (Riprap). This stone will be a reasonably well graded material having a maximum size of 700 pounds. The gradation shall be as follows and shall be within the limits shown on Figure 1 at the end of this section.

Stone Size in Pounds	Percent Lighter by Weight
250-700	100
135-250	50
40-100	15
1-80	5

c. Type B Stone. (Riprap). This stone will be a reasonably well-graded material having a maximum size of 300 pounds. The gradation shall be as follows and shall be within the limits shown on Figure 2 at the end of this section.

Stone Size in Pounds	Percent Lighter by Weight
110-300	100
60-150	50
15-50	15
1-35	5

d. Type C Stone (Spalls). This material will consist of a reasonably well-graded stone and shall have sizes ranging between 8 inches and fines. The gradation shall be as follows and shall be within the limits shown on Figure 3 at the end of this section.

Sieve Designation U.S. Standard Square Mesh	Percent Finer by Weight
8 inches	100
6 inches	80-100
3 inches	40-70
1 inch	0-25
1/2 inch	0-10

e. Type D Stone (Bedding). This material will consist of a reasonably well-graded stone ranging between 4 inches and fines. The gradation shall be as follows and shall be within the limits shown on Figure 3 at the end of this section.

c. Type B Stone. (Riprap). This stone will be a reasonably well-graded material having a maximum size of 300 pounds. The gradation shall be as follows and shall be within the limits shown on Figure 2 at the end of this section.

Stone Size in Pounds	Percent Lighter by Weight
110-300	100
60-150	50
15-50	15
1-35	5

d. Type C Stone (Spalls). This material will consist of a reasonably well-graded stone and shall have sizes ranging between 8 inches and fines. The gradation shall be as follows and shall be within the limits shown on Figure 3 at the end of this section.

Sieve Designation U.S. Standard Square Mesh	Percent Finer by Weight
8 inches	100
6 inches	80-100
3 inches	40-70
1 inch	0-25
1/2 inch	0-10

e. Type D Stone (Bedding). This material will consist of a reasonably well-graded stone ranging between 4 inches and fines. The gradation shall be as follows and shall be within the limits shown on Figure 3 at the end of this section.

Sieve Designation U.S. Standard Square Mesh	Percent Finer by Weight
4 inches	100
2 inches	65-100
1 inch	50-90
3/4 inch	45-83
No.4	25-60
No. 10	14-48
No. 40	0-30
No. 200	0-10

5. Required gradations generally are not standard production items. Some stone materials have a broad gradation band and most producers indicate little or no trouble producing these materials. However, sources that produce coarse aggregates for concrete may have trouble manufacturing or grading materials for the bedding. Contractors will be required to provide the selected sources adequate lead time to produce the various stone products, and the Contractor may propose more than one source for each of the materials.

6. Material Weight. The required minimum specific gravity for this project and Design Analysis level is 2.4 (or 150 pounds per cubic foot) for all materials.

7. Material Quality.

a. General. Quality requirements for each material type are discussed below. Riprap and larger spalls have been subjected to tests established by the Ohio River Division Laboratories, Cincinnati, OH. Tests No. P-11, "Riprap and Breakwater Stone Evaluation" includes a suite of tests to determine stone durability. The smaller size materials such as the smaller spalls and the bedding are included in ORDL Test Nos. C-21 and C-22, (Elementary Acceptance Tests for Fine Aggregates (C-21) and Coarse Aggregates (C-22) for Civil Works."

b. Design Criteria. Design criteria is a limiting factor on the number of available sources. Some producers will be eliminated from the list because their stone failed to meet the minimum specific gravity (SSD) of 2.4.

c. Type A Stone (Riprap, 1 to 700 Pounds). These stones will be a durable material, free from cracks, seams and overburden spoil. Only those sources from which the samples did not show any significant breakdown during the freeze-thaw and wet-dry tests are suitable. The freeze-thaw tests were performed for 35 cycles and the wet-dry tests for 80 cycles.

d. Type B Stone. (Riprap, 1 to 300 pounds). These stones will be a durable material, free from cracks, seams and overburden spoil. Only those sources from which the samples did not show any significant breakdown during the freeze-thaw and wet-dry tests are suitable. The freeze-thaw tests were performed for 35 cycles and the wet-dry tests for 80 cycles.

e. Type C Stone. (Spalls, 1/2 inch - 8 inches). These stones will be a reasonably durable, clean material free from cracks, seams, overburden spoil, and other deleterious materials. Only those sources from which the samples did not show any significant breakdown or deterioration during the freeze-thaw, wet-dry, and ORD lab test No. C-22 tests are suitable.

f. Type D Stone. (Bedding Material, No. 200 Sieve to 4 inches). This material will be a reasonably durable stone, clean and free from overburden spoil, shale, siltstone and other deleterious materials. Only those sources that did not show any significant deterioration in the ORD Lab Test Nos. C-21 or C-22 are suitable.

POSSIBLE SOURCES

8. The required stone materials to construct the facilities can be produced from the sources indicated on plates 1 through 7, "Possible Sources." These sources may be revised for the plans and specifications. However, all material from those sources may not be suitable. The right will be reserved in the specification to reject materials from certain localized areas, zones, strata, channels, or stockpiles when such materials are determined as unsuitable.

9. It is anticipated that selective quarrying will be required for some material types. Blasting techniques used for normal production will require adjustments or in some cases complete tailoring to produce riprap. The specifications shall state that the Contractor require the source to designate lifts, beds, and/or areas of the quarry for the production of riprap. Seasonal blasting and stockpiling of materials will be required prior to delivery at the project. Also, the specifications will require that shale and other undesirable materials will be excluded by adequate processing. All sources proposed by the Contractor will be subject to retesting prior to use in the project.

10. Twenty (20) sources are capable of producing the required materials. Transportation and logistics may be a problem for some of the smaller quarry operators as railheads and loading docks are some miles from the

quarry. Truckers often are reluctant to transport larger materials due to damage of truck beds.

11. Riprap.

a. Type A (1-700 pounds). Nine sources are listed. Three of these are within 32 miles of the project.

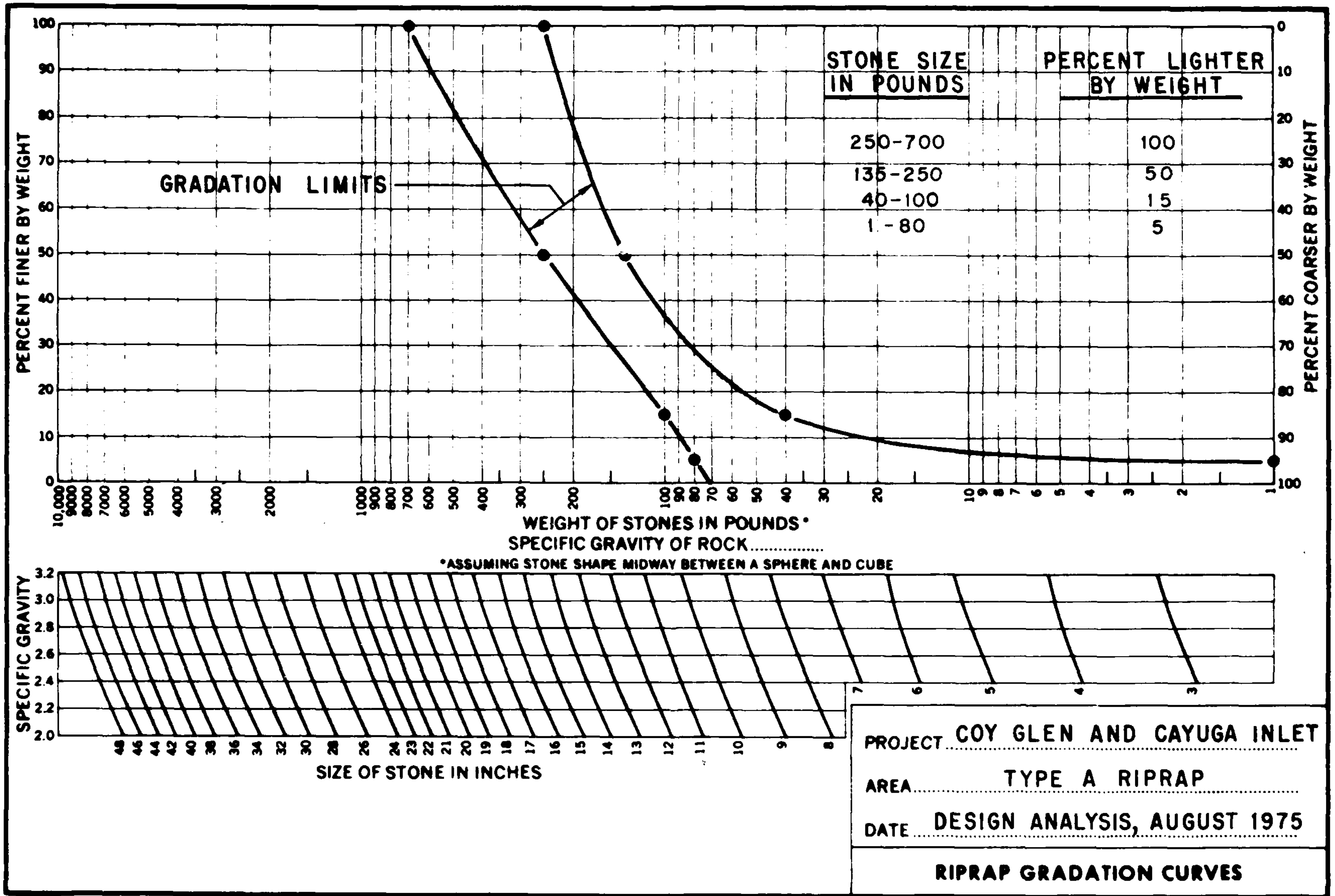
b. Type B (1-300 Pounds). Eleven sources are listed. Three of these are within 32 miles of the project.

12. Spalls. (Type C, 1/2 - 8 inches). Fifteen sources are listed. Three of these are within 32 miles of the project.

13. Bedding Material. (Type D, No. 200-4 inches sieve). Nineteen sources are listed. Three of these are within 32 miles of the project.

14. Riprap was used for both the Cayuga Inlet and Wellsville Rectification Projects. Cayuga Crushed Stone supplied stone to Cayuga Inlet in 1965, 1967, and 1968. Brown Quarry was opened in 1968 to supply additional stone. General Crushed Stone Inc., Honeoye, supplied riprap to the Wellsville Rectification Project in 1971. Only specific ledges in some quarries can produce the required size for riprap. For example, the basal 4 feet at Brown Quarry is too thin-bedded for use as a riprap material. Some quarries will require selective quarrying and productivity may be a problem.

15. Both spalls and bedding gradations are not standard production items and producers will be required to change screens or to blend available gradations.



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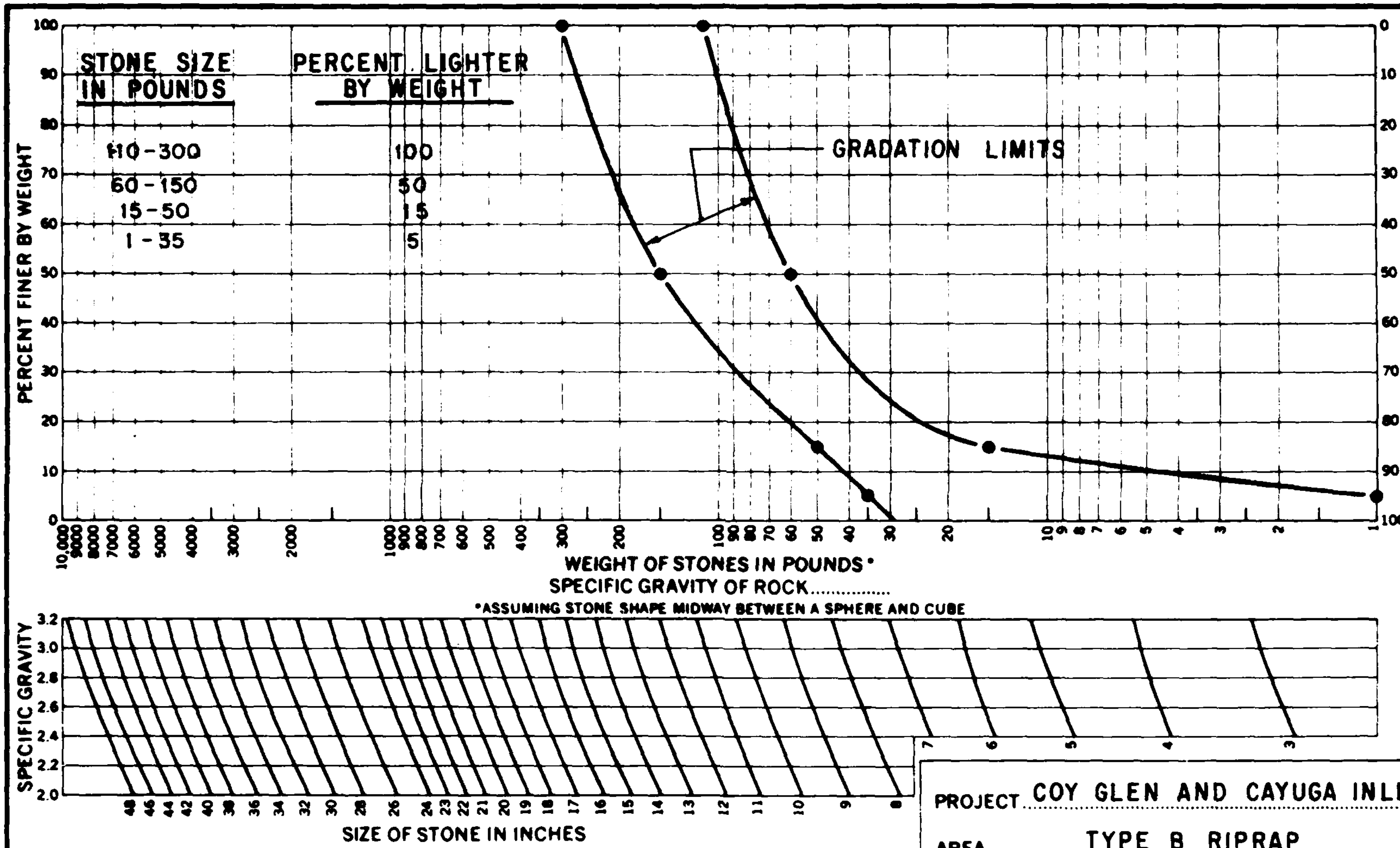
APR 67

FIGURE 1



PROJECT COY GLEN AND CAYUGA INLET

AREA TYPE B RIPRAP

DATE DESIGN ANALYSIS, AUGUST 1975

RIPRAP GRADATION CURVES

ENG FORM 4055
APR 67

FIGURE 2

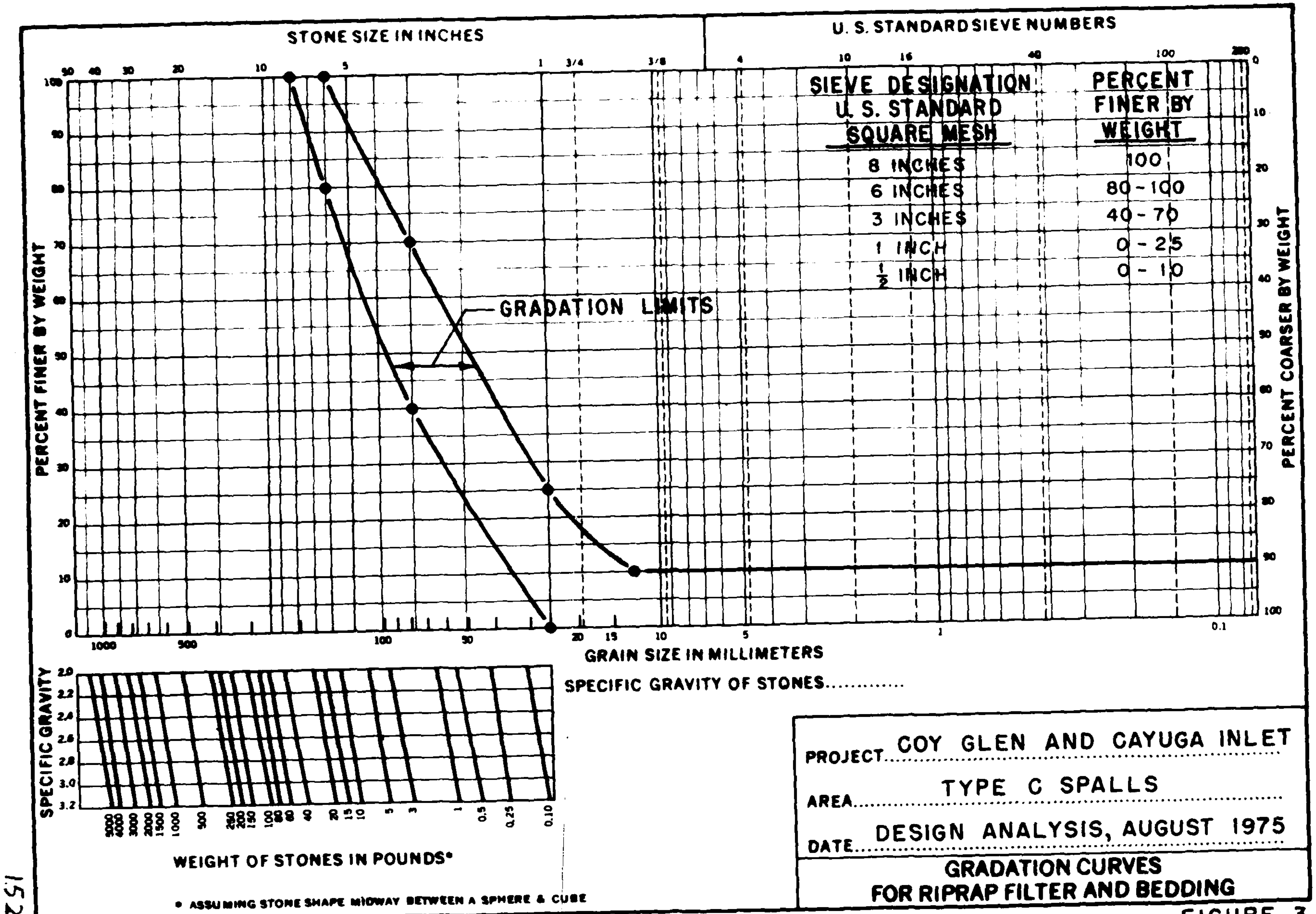
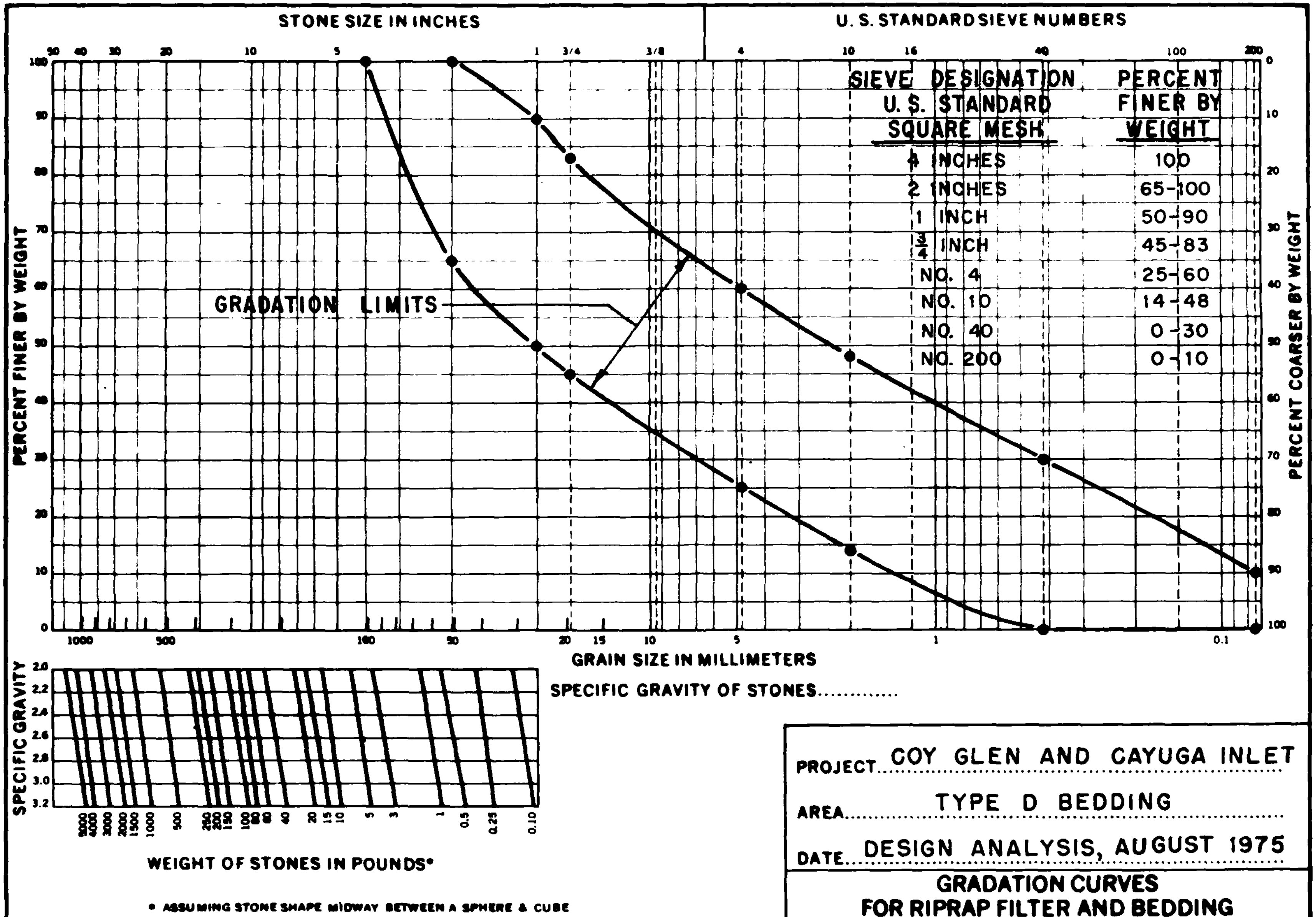


FIGURE 3

ENG FORM 4056
APR 67



SIEVE DESIGNATION U.S. STANDARD SQUARE MESH	PERCENT FINER BY WEIGHT
4 INCHES	100
2 INCHES	65-100
1 INCH	50-90
$\frac{3}{4}$ INCH	45-83
NO. 4	25-60
NO. 10	14-48
NO. 40	0-30
NO. 200	0-10

SPECIFIC GRAVITY OF STONES............

PROJECT COY GLEN AND CAYUGA INLET

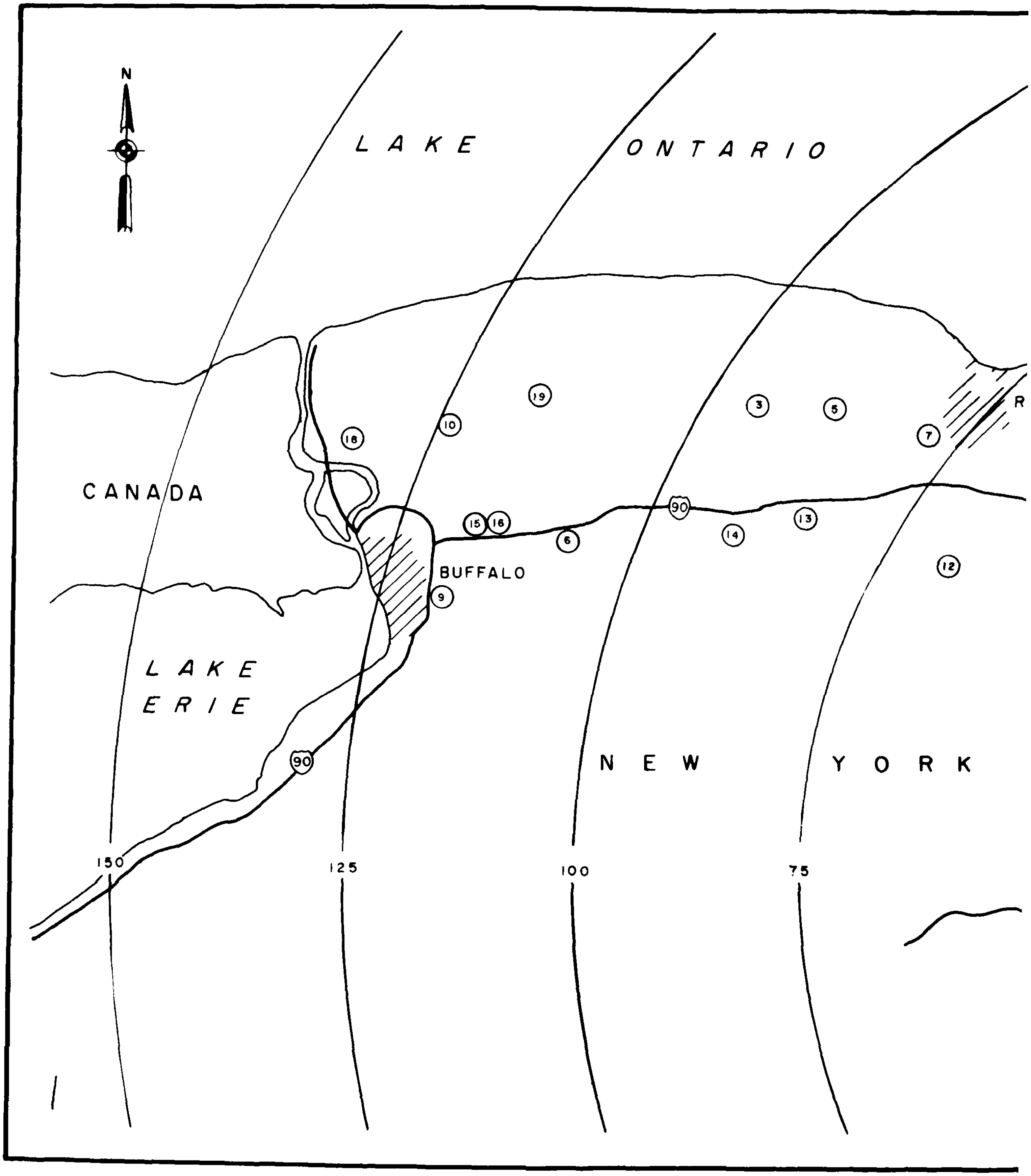
AREA TYPE D BEDDING

DATE DESIGN ANALYSIS, AUGUST 1975

GRADATION CURVES FOR RIPRAP FILTER AND BEDDING

ENG FORM 4056 APR 67

FIGURE 4


N
LAKE ONTARIO
CANADA
LAKE ERIE
BUFFALO
NEW YORK
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19
3
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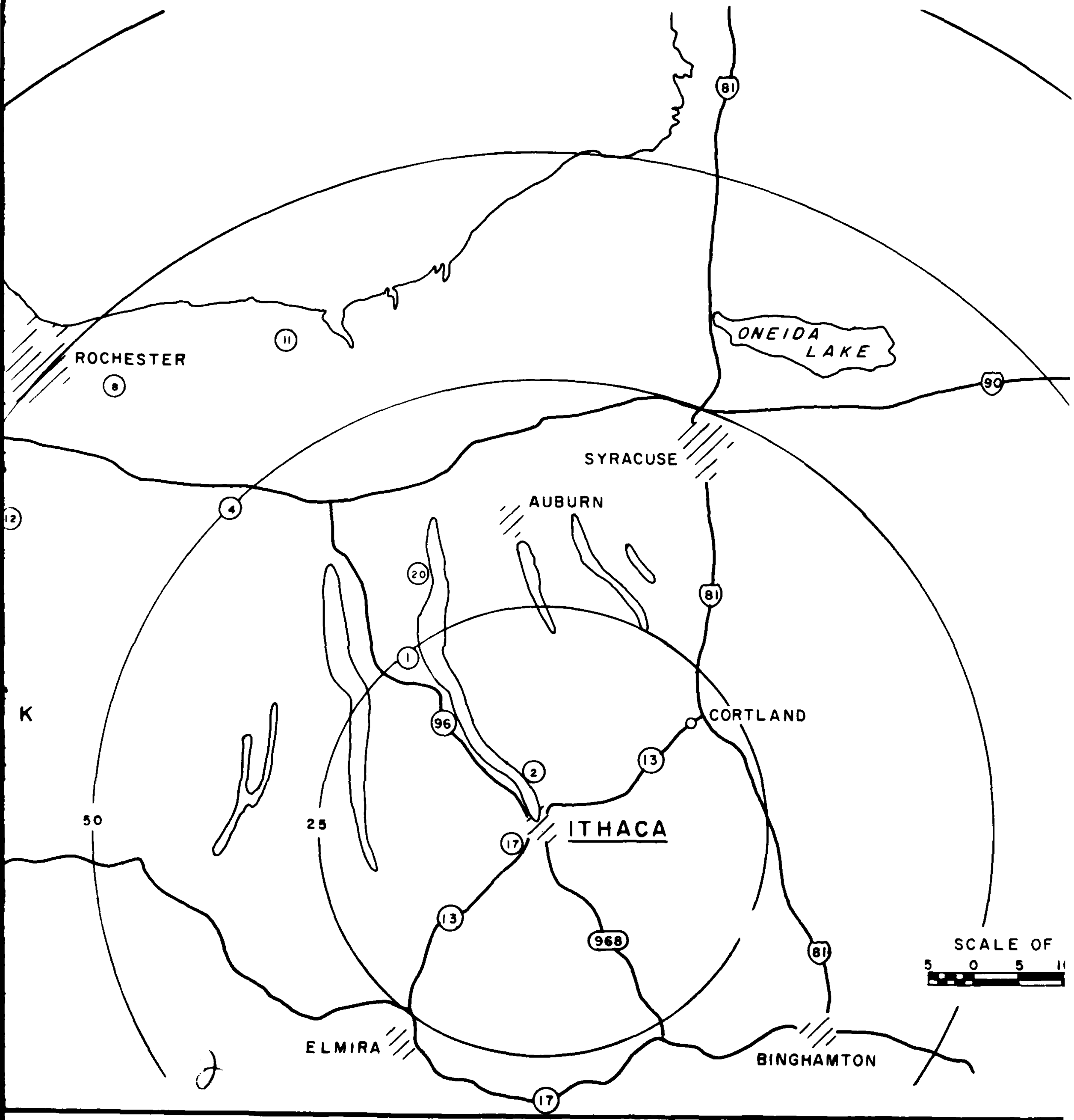
150
125
100
75



ROCHESTER
SYRACUSE
ONEIDA LAKE
AUBURN
CORTLAND
ITHACA
ELMIRA
BINGHAMTON
81
90
11
8
4
12
20
1
96
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13
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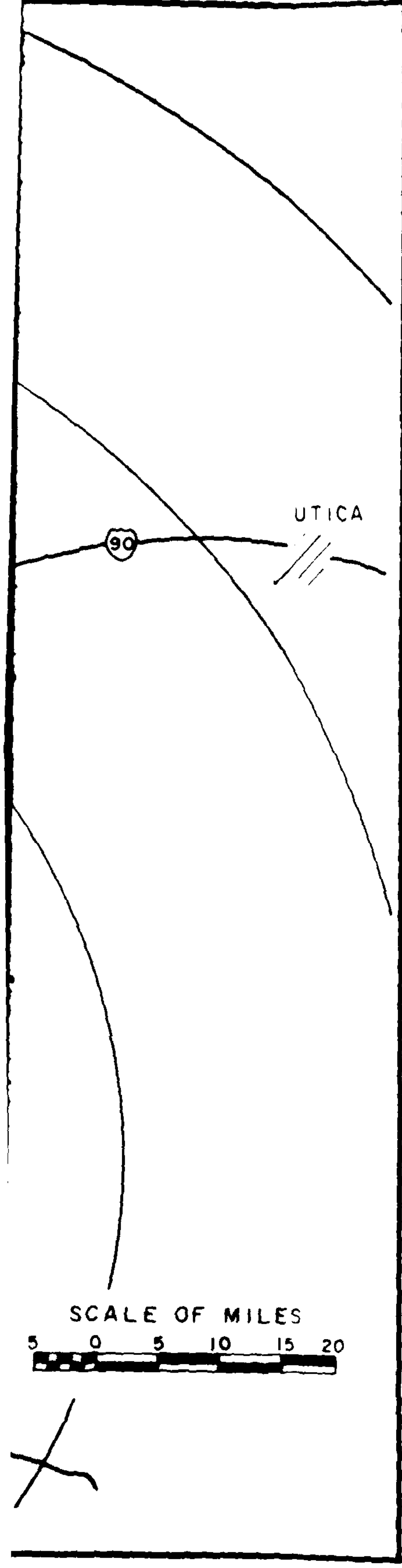

NOTES:

1. NUMBER IN CIRCLE INDICATES QUARRY SITE.
2. FOR QUARRY NAMES AND PRODUCTS, SEE SUPPLEMENT SHEET.

COY GLEN AND CAYUGA INLET
ITHACA, NEW YORK
ENERGY DISSIPATOR FACILITIES
AND RIPRAP REPAIR

MATERIAL SURVEY

U.S. ARMY ENGINEER DISTRICT, BUFFALO
TO ACCOMPANY DESIGN ANALYSIS,
AUGUST, 1975

PLATE I

MAP SUPPLEMENT SHEET
SUMMARY OF POSSIBLE SOURCES FOR
CONSTRUCTION MATERIALS

SITE NUMBER	SOURCE	QUARRY OR PIT LOCATION	RADIAL DISTANCE (IN MILES)	TYPE A RIPRAP	TYPE B RIPRAP	TYPE C SPALLS	TYPE D BEDDING						
1	BROWN QUARRY	OVID, N.Y.	23	X	X	X							
2.	CAYUGA CRUSHED STONE CO.	SOUTH LANSING, N.Y.	6	X	X	X	X						
3.	CLARENDON STONE PRODUCTS	CLARENDON, N.Y.	95				X						
4.	CONCRETE MATERIALS, INC.	MANCHESTER, N.Y.	49			X	X						
5.	CONCRETE MATERIALS, INC.	SWEDEN, N.Y.	88			X	X						
6.	COUNTY LINE STONE CO.	AKRON, N.Y.	107	X	X	X	X						
7.	DOLOMITE PRODUCTS, INC.	GATES CENTER, N.Y.	78		X	X	X						
8.	DOLOMITE PRODUCTS, INC.	PENFIELD, N.Y.	[illegible]8		X	X	X						
9.	FEDERAL CRUSHED STONE CO.	CHEEKTOWAGA, N.Y.	119	X	X	X	X						
10.	FRONTIER STONE PRODUCTS	LOCKPORT, N.Y.	123	X	X	X	X						
11.	GENERAL CRUSHED STONE CO.	SODUS, N.Y.	[illegible]				X						
12.	GENERAL CRUSHED STONE CO.	HONEOYE, N.Y.	[illegible]		[illegible]	[illegible]	X						
13.	GENERAL CRUSHED STONE CO.	LEROY, N.Y.	84				X						
14.	GENESEE STONE PRODUCTS	STAFFORD, N.Y.	90			X	X						
15.	HOUDAILLE CONST. MTLS.	CLARENCE, N.Y.	116	X	X	X	X						
16.	LANCASTER STONE PRODUCTS	CLARENCE, N.Y.	115	X	X	X	X						
17.	LANDSTROM GRAVEL	ITHACA, N.Y.	3				X						
18.	NIAGARA STONE DIVISION	NIAGARA FALLS, N.Y.	132	X	X	X	X						
19.	ROYALTON STONE PRODUCTS	GASPORT, N.Y.	116			X	X						
20.	WARREN BROS.	CANOGA, N.Y.	32	X	X	X	X						

19.	ROYALTON STONE PRODUCTS	GASPORT, N.Y.	6			X	X						
20.	WARREN BROS.	CANOGA, N.Y.	32	X	X	X	X						

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NOTES:

TYPE A RIPRAP - 1 LB. TO 700 POUNDS

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NOTES:

TYPE A RIPRAP - 1 LB. TO 700 POUNDS

TYPE B RIPRAP - 1 LB. TO 300 POUNDS

TYPE C SPALLS - 1/2 IN. TO 8 INCHES

TYPE D BEDDING - NO. 200 SIEVE TO 4 INCHES.

X - INDICATES THAT QUARRY OR PIT IS CAPABLE OF PRODUCING THAT MATERIAL.

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ENERGY DISSIPATOR FACILITIES
AND RIPRAP REPAIR

LOCATION MAP INDEX
POSSIBLE MATERIAL SOURCES

U.S. ARMY ENGINEER DISTRICT, BUFFALO

TO ACCOMPANY DESIGN ANALYSIS, AUGUST 1975

PLATE 2

POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE				
SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	DA
BROWN QUARRY QUARRY NEAR OVID, N.Y. OFFICE NEAR OVID, N.Y.	TULLY LIMESTONE	TYPES A,B AND C STONE	23 MI.	MARCH
CAYUGA CRUSHED STONE CO., INC. QUARRY AT SOUTH LANSING, N.Y. OFFICE AT SOUTH LANSING, N.Y.	TULLY LIMESTONE	TYPES A,B,C AND D STONE	6 MI.	MARCH
				SEPTEM
CLARENDON STONE PRODUCTS QUARRY AT CLARENDON, N.Y. OFFICE AT CLARENDON, N.Y.	LOCKPORT DOLOMITE	TYPE D STONE	95 MI.	MAY 19
CONCRETE MATERIALS, INC. QUARRY AT SWEDEN, N.Y. OFFICE AT SWEDEN, N.Y.	LOCKPORT DOLOMITE	TYPES C AND D STONE	88 MI.	JANUAR

DIAL	LABORATORY TEST RECORD			
TANCE	DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	DATE USED
MI.	MARCH 1969	ORD LAB LAB # 103/69.611C	CAYUGA INLET ITHACA, N.Y.	1969
MI.	MARCH 1967	ORD LAB LAB # 101/67.358C	CAYUGA INLET FLOOD PROTECTION PROJECT, ITHACA, N.Y.	1968
	SEPTEMBER 1965	ORD LAB LAB # 103/66.600C	CAYUGA INLET, STAGES I AND II	1965, 1967 AND 1968
MI.	MAY 1972	ORD LAB LAB # 103/72.610C	OAK ORCHARD HARBOR, N.Y.	UNKNOWN
MI.	JANUARY 1971	ORD LAB LAB # 101/71.362C	ROCHESTER HARBOR, N.Y. EAST PIER REPAIRS	1971
	7			

SERVICE RECORD			
SED	PROJECT	EVALUATION	REM
	CAYUGA INLET ITHACA, N.Y.	UNKNOWN	TESTING REQUIRED. SPECIFIC GRAVIT PROCESSING EQUIPMENT MAYBE LACKIN
	CAYUGA INLET FLOOD PROTECTION PROJECT, ITHACA, N.Y.	SATISFACTORY	SPECIFIC GRAVITY IS 2.76.
) 1968	CAYUGA INLET STAGES I,II AND III	SATISFACTORY	UNIT WEIGHT AVERAGES 171.2 P.C.F. REQUIRED.
	UNKNOWN	UNKNOWN	SPECIFIC GRAVITY IS 2.76. ONLY TR
	ROCHESTER HARBOR, N.Y. EAST PIER REPAIRS	SATISFACTORY	SPECIFIC GRAVITY IS 2.75.
	3		

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ON	REMARKS
	TESTING REQUIRED. SPECIFIC GRAVITY OF 2.72 BASAL 4 FEET NOT ACCEPTABLE PROCESSING EQUIPMENT MAYBE LACKING.
	SPECIFIC GRAVITY IS 2.76.
	UNIT WEIGHT AVERAGES 171.2 P.C.F. RAIL FACILITIES AVAILABLE. TESTING REQUIRED.
	SPECIFIC GRAVITY IS 2.76. ONLY TRUCKING FACILITIES AVAILABLE.
	SPECIFIC GRAVITY IS 2.75.

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U. S. ARMY ENGINEER DISTRICT, BUFFALO
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POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE

SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	D
CONCRETE MATERIALS INC. QUARRY AT MANCHESTER, N.Y. OFFICE AT BROCKPORT, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES C AND D STONE	49 MI.	AUGU
COUNTY LINE STONE CO. QUARRY AT AKRON, N.Y. OFFICE AT AKRON, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	107 MI.	MAY
				FEBR
				SEPT
DOLOMITE PRODUCTS QUARRY AT GATES CENTER, N.Y. OFFICE AT ROCHESTER, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES B,C AND D STONE	78 MI.	MAY
DOLOMITE PRODUCTS QUARRY AT PENFIELD, N.Y. OFFICE AT PENFIELD, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES B,C, AND D STONE	68 MI.	UNKN
				JUNE

RADIAL DISTANCE	LABORATORY TEST RECORD			
	DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	DATE USED
49 MI.	AUGUST 1973	ORD LAB LAB # 103/73.630C	CONFINED DREDGE SPOIL DISPOSAL PROGRAM (RIPRAP)	UNKNOWN
107 MI.	MAY 1967	ORD LAB LAB # 103/67.605C	WARSAW N.Y. FLOOD CONTROL PROJECT (RIPRAP)	1967
	FEBRUARY 1971	ORD LAB LAB # 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	1971
	SEPTEMBER 1974	ORD LAB	CONFINED DREDGE SPOIL DISPOSAL AREAS NOS. 1 AND 2, BUFFALO HARBOR NEW YORK (REPAIRS)	
78 MI.	MAY 1972	ORD LAB LAB # 103/72.610C	OAK ORCHARD HARBOR, N.Y. (CORE STONE, COVER STONE AND CONCRETE AGGREGATE)	UNKNOWN
68 MI.	UNKNOWN	UNKNOWN	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN
	JUNE 1973	ORD LAB LAB # 103/73.603C	CONFINED DREDGE SPOIL DISPOSAL PROGRAM	UNKNOWN

SERVICE RECORD			RE
USED	PROJECT	EVALUATION	
	UNKNOWN	UNKNOWN	UNIT WEIGHT IS 167 P.C.F. TESTI
	WARSAW N.Y., FLOOD CONTROL PROJECT (RIPRAP)	APPEARS SATISFACTORY	THE SECOND LIFT ONLY IS APPROVED MEMBER OF THE ONONDAGA FORMATION TESTING.
	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	TOO EARLY TO EVALUATE	ONLY THE SECOND LIFT, EAST FACE AVERAGES 168 P.C.F. RAIL FACILIT
			BOTH FIRST AND SECOND LIFTS REQ REQUIRED.
	UNKNOWN	UNKNOWN	ONLY THE FIRST LIFT (PENFIELD ME UNIT WEIGHT IS APPROXIMATELY 17 QUARRY. TESTING REQUIRED.
	UNKNOWN	UNKNOWN	ONLY THE PENFIELD MEMBER ACCEPT NOT AVAILABLE.
	UNKNOWN	UNKNOWN	UNIT WEIGHT VARIES FROM 163 P.C. WILL REQUIRE TESTING.
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	REMARKS
	UNIT WEIGHT IS 167 P.C.F. TESTING REQUIRED.
	THE SECOND LIFT ONLY IS APPROVED FOR RIPRAP AND IS FROM THE MOOREHOUSE MEMBER OF THE ONONDAGA FORMATION. CRUSHED MATERIALS WILL REQUIRE TESTING.
	ONLY THE SECOND LIFT, EAST FACE TESTED FOR THIS PROJECT. UNIT WEIGHT AVERAGES 168 P.C.F. RAIL FACILITIES NOT AVAILABLE.
	BOTH FIRST AND SECOND LIFTS REQUIRE RETESTING. SELECTIVE QUARRYING REQUIRED.
	ONLY THE FIRST LIFT (PENFIELD MEMBER) ACCEPTABLE FOR THIS PROJECT. UNIT WEIGHT IS APPROXIMATELY 171 P.C.F. RAIL FACILITIES AVAILABLE AT QUARRY. TESTING REQUIRED.
	ONLY THE PENFIELD MEMBER ACCEPTABLE FOR THIS PROJECT. RAIL FACILITIES NOT AVAILABLE.
	UNIT WEIGHT VARIES FROM 163 P.C.F. TO 171 P.C.F. ALL CRUSHED MATERIALS WILL REQUIRE TESTING.

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POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE

SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	
FEDERAL CRUSHED STONE DIV. OF BUFFALO SLAG CO. INC., QUARRY AT CHEEKTOWAGA N.Y., OFFICE AT BUFFALO N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	119 MI.	NOV
				FEB
				MAR
				APR
FRONTIER STONE PRODUCTS, INC. QUARRY AT LOCKPORT, N.Y. OFFICE AT LOCKPORT, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES A,B,C AND D STONE	123 MI.	FEB
				AUG
GENERAL CRUSHED STONE INC. QUARRY AT SODUS, N.Y. OFFICE AT EASTON, PA.	LOCKPORT FORMATION (DOLOMITE)	TYPE D STONE	61 MI.	MAY
				FEB
				JUN
				JAN

DIAL ANCE	LABORATORY TEST RECORD			
	DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	DATE USED
MI.	NOVEMBER 1965	ORD LAB LAB # 103/66.605C	LOCAL FLOOD PROTECTION PROJECT, SMOKES CREEK, STAGE II (RIPRAP)	UNKNOWN
	FEBRUARY 1971	ORD LAB LAB # 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN
	MARCH 1972	ORD LAB LAB # 103/72.606C	CONFINED DIKE DISPOSAL PROGRAM (CONCRETE AGGREGATE)	UNKNOWN
	APRIL 1973	ORD LAB LAB # 103/73.337C	BLACK ROCK LOCK REHABILITATION	MAY 1973
MI.	FEBRUARY 1971	ORD LAB LAB # 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN
	AUGUST 1974	UNKNOWN	CONFINED DIKE DISPOSAL PROGRAM, BUFFALO HARBOR, N.Y., SITE 4 (ARMOR STONE)	UNKNOWN
MI.	MAY 1971	ORD LAB LAB # 101/71.358C	LITTLE SODUS BAY, N.Y. PIER REPAIR (CONCRETE AGGREGATE)	UNKNOWN
	FEBRUARY 1972	ORD LAB LAB # 103/7[illegible].607C	LITTLE SODUS BAY, N.Y. PIER REPAIR (CONCRETE AGGREGATE)	UNKNOWN
	JUNE 1973	ORD LAB LAB # 103/73.630C	CONFINED DIKE DREDGE DISPOSAL PROGRAM (RIPRAP)	UNKNOWN
	JANUARY 1974	ORD LAB LAB # 103/74.613C	LITTLE SODUS BAY, N.Y. PIER REPAIR (CONCRETE AGGREGATE)	UNKNOWN
	2			

SERVICE RECORD				
USED	PROJECT	EVALUATION	F	
	UNKNOWN	UNKNOWN	UNIT WEIGHT AVERAGES 168 P.C.F	
	UNKNOWN	UNKNOWN	ONLY THE FIRST LIFT. WEST QUA P.C.F. TO 169 P.C.F. RAIL FA	
	UNKNOWN	UNKNOWN	SPECIFIC GRAVITY VARIES FROM 2	
	BLACK ROCK LOCK REHABILITATION	SOME POPOUTS AND SPALLING (1975)	TYPE II, LOW ALKALI CEMENT RE	
	UNKNOWN	UNKNOWN	THE DECEW MEMBER NOT ACCEPTAB FROM 162 P.C.F. RAIL FACILIT	
	UNKNOWN	UNKNOWN	ONLY THE GASPORT MEMBER ACCEP ON NYS BARGE CANAL TO BE AVAI DECEW MEMBER CURRENTLY BEING WILL REQUIRE TESTING.	
	UNKNOWN	UNKNOWN	ALL CRUSHED MATERIALS WILL RE	
	UNKNOWN	UNKNOWN		
	UNKNOWN	UNKNOWN		
	UNKNOWN	UNKNOWN		EI
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	REMARKS
	UNIT WEIGHT AVERAGES 168 P.C.F.
	ONLY THE FIRST LIFT, WEST QUARRY TESTED. UNIT WEIGHT VARIES FROM 166 P.C.F. TO 169 P.C.F. RAIL FACILITIES NOT AVAILABLE.
	SPECIFIC GRAVITY VARIES FROM 2.68 TO 2.70. LOW ALKALI CEMENT REQUIRED.
LING	TYPE II, LOW ALKALI CEMENT REQUIRED.
	THE DECEW MEMBER NOT ACCEPTABLE FOR THIS PROJECT. UNIT WEIGHTS VARY FROM 162 P.C.F. RAIL FACILITIES NOT AVAILABLE.
	ONLY THE GASPORT MEMBER ACCEPTABLE FOR ARMOR STONE. LOADING FACILITIES ON NYS BARGE CANAL TO BE AVAILABLE. SELECTIVE QUARRYING REQUIRED. DECEW MEMBER CURRENTLY BEING RETESTED (JULY 1975). CRUSHED MATERIALS WILL REQUIRE TESTING.
	ALL CRUSHED MATERIALS WILL REQUIRE RETESTING.

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POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE				
SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	
GENERAL CRUSHED STONE CO. QUARRY AT HONEOYE FALLS, N.Y OFFICE AT EASTON, PA.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	68 MI.	DEC
GENERAL CRUSHED STONE CO. QUARRY AT LEROY, N.Y. OFFICE AT EASTON, PA.	ONONDAGA FORMATION (LIMESTONE)	TYPE D STONE	84 MI.	DEC
GENESEE STONE PRODUCTS CORP. QUARRY AT STAFFORD, N.Y. OFFICE AT BATAVIA, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES C AND D STONE	90 MI.	DEC
				JAN
HOUDAILLE CONSTRUCTION MATERIALS, INC. QUARRY AT CLARENCE, N.Y. OFFICE AT CLARENCE, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	116 MI.	JUL
				SEP
				FEB
				APR

LABORATORY TEST RECORD				
DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	DATE USED	
ECEMBER 1971	ORD LAB LAB # 103/72.602C	WELLSVILLE RECTIFICATION PROJECT, WELLSVILLE N.Y. (RIPRAP)	1971	WELLSVILLE PROJECT (R
ECEMBER 1971	ORD LAB LAB # 103/72 602C	WELLSVILLE RECTIFICATION PROJECT, WELLSVILLE, N.Y. (RIPRAP)	UNKNOWN	UNKNOWN
ECEMBER 1971	ORD LAB LAB # 103/72.602C	WELLSVILLE RECTIFICATION PROJECT, WELLSVILLE, N.Y. (RIPRAP)	UNKNOWN	UNKNOWN
JANUARY 1974	ORD LAB LAB # 103/74.610C	WELLSVILLE RECTIFICATION PROJECT, WELLSVILLE, N.Y. (RIPRAP)	UNKNOWN	UNKNOWN
JULY 1959	ORD LAB LAB # 412/59Z	NORTH ENTRANCE, BUFFALO HARBOR, N.Y. (CORE STONE)	UNKNOWN	UNKNOWN
SEPTEMBER 1965	ORD LAB LAB # 103/66.602C	LOCAL FLOOD PROTECTION PROJECT, SMOKES CREEK, STAGE II, (RIPRAP)	UNKNOWN	UNKNOWN
FEBRUARY 1971	ORD LAB LAB # 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP AND SPALLS)	1971	BUFFALO D (RIPRAP A
APRIL 1972	ORD LAB LAB # 103/72.606C	CONFINED DREDGE SPOIL DISPOSAL PROGRAM (CONCRETE AGGREGATE)	UNKNOWN	UNKNOWN
2				

	SERVICE RECORD		REMA
ED	PROJECT	EVALUATION	
	WELLSVILLE EMERGENCY FLOOD CONTROL PROJECT (RIPRAP)	SATISFACTORY	QUARRY NOT RESPONSIBLE FOR GRADATI P.C.F. TO 168 P.C.F. RAIL FACILIT CRUSHED MATERIAL WILL REQUIRE TEST
	UNKNOWN	UNKNOWN	UNIT WEIGHT AVERAGES 167 P.C.F. Q UNIFORM SIZE RIPRAP. RAIL FACILIT CRUSHED MATERIALS WILL REQUIRE TEST
	UNKNOWN	UNKNOWN	ONLY THE FIRST AND SECOND LIFT ACC 168 P.C.F. RAIL FACILITIES NOT AV
	UNKNOWN	UNKNOWN	THE THIRD LIFT IS NOT ACCEPTABLE.
	UNKNOWN	UNKNOWN	CRUSHED MATERIALS WILL REQUIRE TEST
	UNKNOWN	TOO THIN BEDDED FOR USE ON PROJECT TESTED FOR	
	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP AND SPALLS)	TOO EARLY TO EVALUATE	ONLY THE SECOND LIFT TESTED AND US P.C.F. TO 171 P.C.F. RAIL FACILIT
	UNKNOWN	UNKNOWN	NOT RECOMMENDED FOR USE AS CONCRET REQUIRED.
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REMARKS
QUARRY NOT RESPONSIBLE FOR GRADATION. UNIT WEIGHT VARIES FROM 166 P.C.F. TO 168 P.C.F. RAIL FACILITIES NOT AVAILABLE. CRUSHED MATERIAL WILL REQUIRE TESTING
UNIT WEIGHT AVERAGES 167 P.C.F. QUARRY WILL NOT PROCESS A GRADED OR UNIFORM SIZE RIPRAP. RAIL FACILITIES AVAILABLE. CRUSHED MATERIALS WILL REQUIRE TESTING.
ONLY THE FIRST AND SECOND LIFT ACCEPTABLE. UNIT WEIGHT AVERAGES 168 P.C.F. RAIL FACILITIES NOT AVAILABLE.
THE THIRD LIFT IS NOT ACCEPTABLE.
CRUSHED MATERIALS WILL REQUIRE TESTING.
ONLY THE SECOND LIFT TESTED AND USED. UNIT WEIGHT VARIES FROM 165 P.C.F. TO 171 P.C.F. RAIL FACILITIES AVAILABLE.
NOT RECOMMENDED FOR USE AS CONCRETE AGGREGATE. LOW ALKALI CEMENT REQUIRED

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POSSIBLE SOURCES FOR TYPES A,B,C AND D STONE

SOURCE	ROCK TYPE	PROPOSED USE	RADIAL DISTANCE	
LANCASTER STONE PRODUCTS CORP. QUARRY AT CLARENCE, N.Y. OFFICE AT WILLIAMSVILLE, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	115 MI.	OCTO
LANDSTROM GRAVEL PIT PIT AT ITHACA, N.Y. OFFICE AT ITHACA, N.Y.	GLACIAL DEPOSIT	TYPE D STONE	3 MI.	UNK
NIAGARA STONE DIV. OF GREAT LAKES COLOR PRINTING CORP., QUARRY AT NIAGARA FALLS, N.Y. (PLETCHERS CORNERS) OFFICE AT NIAGARA FALLS, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES A,B,C AND D STONE	132 MI.	FEBR
ROYALTON STONE PRODUCTS, INC. QUARRY AT GASPORT, N.Y. OFFICE AT GASPORT, N.Y.	LOCKPORT FORMATION (DOLOMITE)	TYPES C AND D STONE	116 MI.	FEBR
WARREN BROS. QUARRY AT CANOGA, N.Y. OFFICE AT GENEVA, N.Y.	ONONDAGA FORMATION (LIMESTONE)	TYPES A,B,C AND D STONE	32 MI.	OCT

LABORATORY TEST RECORD				
DATE TESTED	LABORATORY	PROJECT FOR WHICH TESTED	DATE USED	
OCTOBER 1967	ORD LAB LAB * 103/68.605C	BUFFALO DIKED DISPOSAL AREA #1 (RIPRAP)	UNKNOWN	UNKN
NKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	UNKN
FEBRUARY 1971	ORD LAB LAB * 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN	UNKN
FEBRUARY 1971	ORD LAB LAB * 103/71.612C	BUFFALO DIKED DISPOSAL AREA #2 (RIPRAP)	UNKNOWN	UNKN
OCTOBER 1968	ORD LAB LAB * 103/74.601C	GREAT SODUS HARBOR, N.Y. EMERGENCY WEST PIER REPAIR (BREAKWATER STONE)	UNKNOWN	UNKN
2				

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ENERGY DISSIPATOR FACILITIES AND RIPRAP REPAIR, COY GLEN AND CA--ETC(U)
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SERVICE RECORD		REMARKS
PROJECT	EVALUATION	
UNKNOWN	UNKNOWN	ONLY THE LOWER LIFT TESTED (1967). UNIT W TO 169 P.C.F. RAIL FACILITIES NOT AVAILAB CRUSHED MATERIALS WILL REQUIRE TESTING.
UNKNOWN	UNKNOWN	TESTING REQUIRED. AN ACCEPTABLE SOURCE FO
UNKNOWN	UNKNOWN	BOTH LIFTS CONSISTING OF OAK ORCHARD, ERAM MEMBERS ACCEPTABLE. UNIT WEIGHT VARIES FR RAIL FACILITIES AVAILABLE. MANAGEMENT MAY SIZE MATERIAL, CRUSHED MATERIALS REQUIRE TE
UNKNOWN	UNKNOWN	ONLY MATERIALS FROM EAST END OF QUARRY TES FROM 163 P.C.F. TO 165 P.C.F. RAIL FACILI CRUSHED MATERIALS REQUIRE TESTING
UNKNOWN	UNKNOWN	UNIT WEIGHT VARIES FROM 166 P.C.F. TO 169 CRUSHED MATERIALS REQUIRE TESTING.
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REMARKS
ONLY THE LOWER LIFT TESTED (1967). UNIT WEIGHT VARIES FROM 166 P.C.F. TO 169 P.C.F. RAIL FACILITIES NOT AVAILABLE. CRUSHED MATERIALS WILL REQUIRE TESTING.
TESTING REQUIRED. AN ACCEPTABLE SOURCE FOR CAYUGA INLET, STAGE III.
BOTH LIFTS CONSISTING OF OAK ORCHARD, ERAMOSA AND UPPER GOAT ISLAND MEMBERS ACCEPTABLE. UNIT WEIGHT VARIES FROM 166 P.C.F. TO 174 P.C.F. RAIL FACILITIES AVAILABLE. MANAGEMENT MAY BE RELUCTANT TO PRODUCE LARGE SIZE MATERIAL, CRUSHED MATERIALS REQUIRE TESTING.
ONLY MATERIALS FROM EAST END OF QUARRY TESTED. UNIT WEIGHT VARIES FROM 163 P.C.F. TO 165 P.C.F. RAIL FACILITIES AVAILABLE. CRUSHED MATERIALS REQUIRE TESTING
UNIT WEIGHT VARIES FROM 166 P.C.F. TO 169 P.C.F. CRUSHED MATERIALS REQUIRE TESTING.

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